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VI

Concrete and reinforced concrete in hydraulic engineering (Dams, pipe lines, pressure galleries etc.)

Beton und Eisenbeton im Wasserbau (Staumauern, Rohrleitungen, Druckstollen usw.

Application du béton et du béton armé aux travaux hydrauliques (Barrages, conduites, galeries sous pression, etc.)

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Development of the Analysis of Arch Dams.

Entwicklung der Berechnung von Bogen-Staumauern.

Le développement du calcul des barrages arqués.

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Introduction.

Masonry dams were in the beginning executed straight and calculated as vertical cantilevers between two vertical crossections, fixed in the foundations, loaded with the weight of masonry and water pressure; both loads produce combined compression and bending stresses which cause the strength of masonry to be but little utilised, especially as the tensile strength of masonry is neglected. To eliminate the dangerous effect of temperature changes, a slight curvature of dams was later used; but the stresses were computed as if the dam were straight. It was supposed that the curved dam accomodated itself to the changes produced in its length, which are the consequence of temperature changes, by a change in its curvature. The dam also being fixed at the abutments in a horizontal direction, it was supposed that there was additional safety both for the weight of masonry and for the water pressure. But a detailed statical investigation showed that a slight curvature of the dam has not the favorable consequences expected, because the usual computation gives great thickness. For, if the dam is considered as a horizontal arch under water pressure, the computation gives, with a slight curvature and a great thickness of arch, tensions at the abutments in the extrados and at the crown in the intrados, which can produce vertical cracks in the masonry of the dam.[1] Though the strenghtening of the dam will obviate cracks in horizontal joints, cracks in vertical joints may nevertheless occur; the strengthening of the dam with a surplus of masonry is only apparent as the masonry is not rightly located.

Analysis of Arch Dam as a System of Independent Horizontal Arches.

An arch is, in comparison to a cantilever, a much better structural element as it permits, given a right disposition, a much more uniform distribution of stresses on the masonry and a better use of its strength. The first conscious application of it was made about 1800 in the Meer Allum Dam at Hyderabad in India[2] with 21 horizontal arches between vertical buttresses and in 1845 in the dam built after the project of *M. Zola*[2] near Aix in France in a narrow valley and having the shape of one single horizontal arch.

The following considerations refer as a rule the up-stram face of the dam vertical.

The analysis of arch dams considered, at first approximately, the horizontal arches in different heights as independent arches, loaded with the whole radial water pressure, uniformly distributed along the length of the arch. This method makes no allowance for the mutual connection in the vertical direction; it therefore disregards the shearing stresses in horizontal planes between adjacent arches, which are the consequence of various horizontal displacements. When the reservoir is empty, the weight of the upper arches acts vertically upon the lower arches as in a straight dam; when the reservoir is full, this method considers horizontal elements as independent arches, each of which bears its full water pressure. If the up-stream face is inclined, the vertical component of water pressure adds to the weight of masonry[3]. *Deloore*, who made the first theoretical analysis of arch dams,[4] supposed approximately that the resultant of stresses caussed by water pressure in the crown and abutment joint passes through a point distant by one-third thickness from the up-stream face. *Pelletreas*[5] supposes for uniformly distributed radial water pressure the circular centre line of arch as a pressure line (an for thin cylindrical shells equally loaded), he therefore assumes a uniform pressure in all sections of the arch. This method was then customary, especially in America in the majority of cases, and the many arch dams in Australia were calculated in this manner, which is still advocated by *H. Hawgood*[6]. The dams calculated by this method proved very safe. The transmission of external forces by arch action causes a much better division of stresses and a very considerable diminution of thickness compared with dams opposing to water pressure only the weight of masonry as vertical cantilevers, which are therefore very uneconomical as regards the division of stresses and the utilisation of masonry strength.

R. Ruffieux[7] first calculated the horizontal arch of arch dams as an elastic arch with fixed ends (according to the theory of *J. Résal*), taking also into account the effect of normal stresses, which is very essential here, and using the theorie of the *thin arch*. The same method was used later by *E. Mörsch*[8], *H. Ritter*[9], *C. Guidi*[10], *W. Cain*[11], *R. Kelen*[12] and *G. Ippolito*[13].

In the analysis of an arch dam as a system of independent horizontal arches, the usual assumption was, as for thin cylindrical shells, that the stresses are uniformly distributed throughout the thickness t, that is, the circular centre line was supposed as pressure line to the uniformly distributed radial pressure p_2 on the extrados of arch with radius r_2 (fig. 1). That gives in each section a thrust $N_o = - p_2 r_2$ or a stress

$$v_0 = \frac{N_0}{A} = - \frac{p_2 r_2}{b t}, \tag{1}$$

for an arch of a length b, area of section $A = bt$; the thrust N and the arch stress v are positive, if they are tensions. Instead of pressure p_2 on the extrados, a radial pressure p uniformly distributed along the centre line with radius r can be considered; it is

$$p = p_2 \frac{r_2}{r} \tag{2}$$

This method also corresponds to the analysis of an elastic arch, neglecting the effect of thrust (the shortening of centre line), for then the stress centre coincide with the centre of sections. But a detailed investigation showed that in this case the effect of thrust cannot be neglected even for higher arches. The thrust shortens the centre line, which would change in a circle of shorter radius if the abutments were free; since the width of arch does not change with fixed abutments, the arch cannot remain circular and therefore the centres of stresses must depart from centres of sections. The centre line, deformed by the thrust N_o, if the abutments are assumed to be free, can be brought into shape where the abutments come into their original position, by adding a horizontal force ΔH, going through centre of gravity of the centre line, as an additional reaction acting in both abutments in the outward direction[8] (fig. 1); its value is generally

$$\Delta H = \frac{N_o \int \frac{\cos\varphi \, ds}{A}}{\int \frac{y^2\, ds}{J} + \int \frac{\cos^2\varphi \, ds}{A}} \tag{3}$$

for constant thickness

$$\Delta H = \frac{N_o\, l}{\frac{A}{J}\int y^2\, ds + \int \cos^2\varphi\, ds} = \frac{N_o\, l}{\left(\frac{12\, r^2}{t^2} + 1\right)\left[\frac{l}{2\,r}(r - h) + \frac{s}{2}\right] - \frac{12\, l^2 r^2}{s\, t^2}} \tag{3a}$$

if A = area of section, J = moment of inertia and $s = 2r\,\alpha$ = length of centre line.

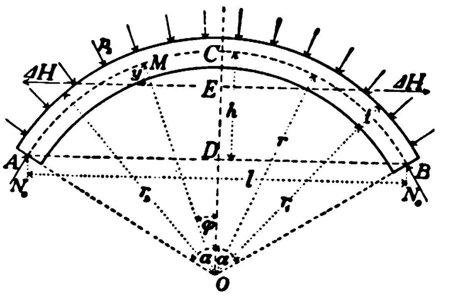

Fig. 1.

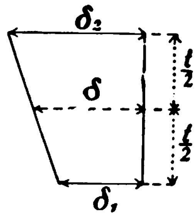

Fig. 2.

The temperature change, equal at all points of arch, gives a horizontal force

$$H_t = \frac{\delta\, \varepsilon\, E\, l}{\frac{1}{J}\int y^2\, ds + \frac{1}{A}\int \cos^2\varphi\, ds} \tag{4}$$

where δ = temperature change, ε = coefficient of temperature expansion, E = modulus of elasticity. If the change of temperature varies linearly from δ_1 at the intrados to δ_2 at the extrados (fig. 2) in all the sections, it produces in the abutments only a bending moment

$$M_t' = -(\delta_2 - \delta_1)\, \varepsilon\, E \, \frac{\int \frac{ds}{t}}{\int \frac{ds}{J}} \tag{5}$$

for constant thickness

$$M_t' = -(\delta_2 - \delta_1)\,\varepsilon\,E\,\frac{J}{t} \tag{5a}$$

A detailed analysis of the case, already made by *H. Ritter*[9] and later by *A. Stucky*[14], showed that also shear has an influence which can be of considerable importance for flat arches. The denominator in the formula (3) for ΔH has a general value

$$\int \frac{y\,y'\,ds}{J} + \int \frac{\cos^2\varphi\,ds}{A} + \beta\frac{E}{G}\int\frac{\sin^2\varphi\,ds}{A}$$

if β = reduction coefficient of shear $\left(\text{for a rectangular section } = \frac{6}{5}\right)$, G = shearing modulus, y' = ordinate of the antipole of the axis of gravity of the centre line with respect to the ellipse of elasticity of the element of arch. For isotropical substances $\frac{E}{G} = 2.5$, therefore $\beta\,\frac{E}{G} = 3$. For thin arches approximately $y' = y$ and for constant thickness the value of ΔH becomes

$$\Delta H = -\frac{p\,r\,t^2}{C_1 r^2 + C_2 t^2};\; C_1 = 6\left(\cos\alpha + \frac{\alpha}{\sin\alpha} - \frac{2\sin\alpha}{\alpha}\right);\; C_2 = \frac{2\,\alpha}{\sin\alpha} + \cos\alpha. \tag{3b}$$

H. Ritter[9] has computed tables for C_1, C_2 which facilitate the calculation. A constant temperature change produces the horizontal force

$$H_t = \frac{\delta\,\varepsilon\,E\,t^3}{C_1 r^2 + C_2 t^2} \tag{4a}$$

acting in the axis of gravity of the centre line. *Ritter* determines the effect of temperature change also in the case when temperature varies in the section continually, after a curve from zero at the extrados to maximum at the intrados. If (reservoir being empty), the temperature change in the section is symmetrical to its centre, the horizontal force H_t has the value (4a), for δ being the mean temperature change in the section.

Very detailed is the analysis of arches under radial loads in the article by *W. Cain*[11] and in the following discussion, further in the article by *F. A. Noetzli*[15] and in the discussion on it. *W. Cain* published in his article and in his conclusion of the discussion[16] the final formulas for calculation of fixed arches under uniformly distributed normal loads (fig. 3), as follows. The thrust H_c at the crown is given by

$$X = p\,r - H_c = \frac{p\,r}{\vartheta}\cdot 2\,\frac{i^2}{r^2}\,\alpha\sin\alpha, \tag{6}$$

where

$$\vartheta = \left(1 + \frac{i^2}{r^2}\right)\alpha\left(\alpha + \frac{1}{2}\sin 2\,\alpha\right) - 2\sin^2\alpha + 2.88\,\frac{i^2}{r^2}\,\alpha\left(\alpha - \frac{1}{2}\sin 2\,\alpha\right); \tag{6a}$$

i = radius of gyration $\left(i^2 = \frac{1}{12}t^2\right)$; the numerical factor $2.88 = \beta\,\frac{E}{G}$ with $\frac{E}{G} = 2.4$ for concrete (instead of $\frac{E}{G} = 2.5$ for isotropical substances) and $\beta = \frac{6}{5}$

for rectangular section. The member with the factor 2.88 comes from shear; the effect of shear can be neglected for central angles $90^0 < 2\alpha < 120^0$, but for smaller central angles and for large proportions t/r the effect of shear can be great enough. For the point M of the arc, given by the angle with the axis of symmetry, the thrust (positive for tension) is

$$N = X \cos \varphi - p\, r, \tag{7}$$

the shear

$$T = X \sin \varphi \tag{8}$$

and the bending moment (positive when clockwise for forces on the left side)

$$M = - X\, r \left(\frac{\sin \alpha}{\alpha} - \cos \varphi\right); \tag{9}$$

that is, the moment to the point M of a force X acting to the right in the centre of gravity E of the centre line, if it is the effect of the right portion, because the centre of gravity E is given by the distance $OE = \frac{r \cdot \sin \alpha}{\alpha}$. These results signify that in each section the force X, acting in the centre of gravity E, adds to the thrust $N_o = - p\, r = - p_2\, r_2$.

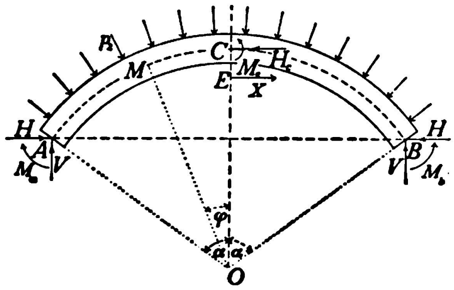

Fig. 3.

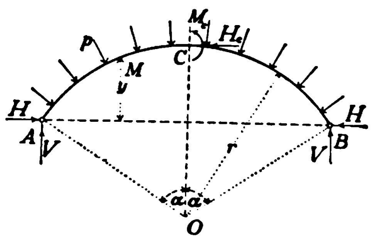

Fig. 4.

The uniform radial loads produce the deflection in the crown of arch (positive toward the centre of arch)

$$\eta = \omega \frac{p\, r^2}{E\, t}; \tag{10}$$

where

$$\omega = \frac{\alpha}{\vartheta}(1 - \cos \alpha) \left[\left(1 + \frac{i^2}{r^2}\right)(\alpha - \sin \alpha) + 2.88 \frac{i^2}{r^2}(\alpha + \sin \alpha) \right] \tag{10a}$$

A temperature change δ, equal at all points of arch, gives a horizontal force

$$H_t = \delta\, \varepsilon \frac{E\, J}{r^2} \cdot \frac{2\, \alpha \sin \alpha}{\vartheta} \tag{11}$$

going through centre of gravity of the centre line, and a deflection of the crown section

$$\eta t = - \omega \cdot \delta\, \varepsilon\, r; \tag{12}$$

ω is the coefficient given by (10a). A good check of the foregoing equations is that for $\alpha = 0$ they become equations for a straight beam fixed at both ends; it follows by substituting infinite series for sin and cos and limiting for $\alpha = 0$.

Is the connection between arch and foundation not rigid (anchoring of reinforcing bars), the arch at the abutments can crack and it approches the arch with two hinges, especially in a thin arch. In this case (fig. 4), neglecting the influence of shear, as is possible with a thin arch, it works out that

$$X = p\,r - H_c = \frac{p\,r}{\vartheta'} \cdot 2 \frac{i^2}{r^2} \sin \alpha, \tag{13}$$

$$\vartheta' = \alpha\,(2 + \cos 2\,\alpha) - \frac{3}{2} \sin 2\,\alpha + \frac{i^2}{r^2} \left(\alpha + \frac{1}{2} \sin 2\,\alpha\right), \tag{13a}$$

$$M = X\,y \tag{14}$$

For N and T we have equations (7), (8). The deflection of the crown of arch is

$$\eta = \omega' \cdot \frac{p\,r^2}{E\,t} \tag{15}$$

$$\omega' = 1 - \frac{\cos \alpha}{\vartheta'} \left[\sin \alpha + \alpha\,(1 - 2 \cos \alpha) + \frac{i^2}{r^2} (\alpha - \sin \alpha)\right] \tag{15a}$$

A temperature change, equal at all points of the arch, produces in the abutments horizontal reactions

$$H_t = \delta\,\varepsilon \cdot \frac{E\,J}{r^2} \cdot \frac{2 \sin \alpha}{\vartheta'} \tag{16}$$

and the deflection of the crown

$$\eta_t = - \omega' \cdot \delta\,\varepsilon\,r. \tag{17}$$

Cam. Guidi[10] transformed the equations for a hingeless arch, introducing lengths instead of goniometrical functions. To the thrust $N = - p_2 r_2 = - p\,r$ in all sections there comes in both abutments an additional horizontal reaction going through the centre of gravity of centre line; its value is (see fig. 1)

$$\Delta H = - \frac{p\,r}{\vartheta''} \cdot 2 \frac{i^2}{r^2}; \tag{18}$$

$$\vartheta'' = \frac{s}{l} + \frac{r - h}{r} - \frac{2\,l}{s} + 2 \frac{i^2}{r^2} \left(2 \frac{s}{l} - \frac{r - h}{r}\right) \tag{18a}$$

The result represents the effects of the bending moment, the thrust and the shear with $\beta \frac{E}{G} = 3$ (as for isotropical substances). An equal temperature change δ at all points of arch gives

$$H_t = \frac{\delta\,\varepsilon\,E\,t^3}{6\,\vartheta''\,r^2} \tag{11a}$$

acting in the axis of gravity of centre line. A uniform radial water pressure produces the deflection of the crown of arch

$$\eta = \frac{p\,r}{E\,t} h \left\{1 + \frac{1}{\vartheta''} \left[2 \frac{l}{s} - \frac{l^2}{4\,h\,r} \left(1 - 2 \frac{i^2}{r^2}\right)\right]\right\} \tag{10b}$$

The deflection produced by a constant temperature change is

$$\eta_t = \eta \cdot \frac{\delta \varepsilon E t}{p r}; \tag{12a}$$

this coincides with the equation (12) of *Cain. Guidi* facilitates the calculations by means of numerous tables giving for different values of the central angle 2α the values of $\frac{s}{r}$, $\frac{s}{l}$, $\frac{l}{s}$, $\frac{l}{2r}$, $\eta : \frac{pr}{E}$. He also analyses the non-uniform water pressure, which appears with inclined axes (surface lines) of arches in multiple-arch dams, and the effect of dead load for an arch with inclined axis, the arch of variable section and the butresses of multiple-arch dams. *H. Ritter*[9] analyses the arch of general form and with variable section.

A rapid preliminary calculation can be based on simple formulas given by *F. A. Noetzli*[17]. He neglects the effect of thrust and shear, replaces the centre line approximately with a parabola and neglects the difference between the length of arc and chord, assuming a low arch; thus he gets

$$\Delta H = -0.94\, p_2 r_2 \frac{t^2}{h^2}. \tag{19}$$

More accurate would be, instead of 0.94, the coefficient

$$k_t = \frac{h^2 l}{t^3 \left(\int \frac{y^2 ds}{J} + \int \frac{ds}{A}\right)}; \tag{19a}$$

its values are given by *Noetzli* for various central angles and for various proportions t/h in a diagram. The coefficient k_t is not yet exact, but it considers the thrust and the shear (with approximation, using 1 instead of $\beta \frac{E}{G} = 3$); it gives values very near to the exact ones, as *W. A. Miller*[18] proved. *Noetzli* gives for effect of temperature the approximate formula

$$H_t = 0.94\, \delta \varepsilon E \frac{t^3}{h^2} \tag{20}$$

on the same basis as equation (19); he supposes approximately H_t acting at a distance of $\frac{h}{3}$ from the crown of centre line as for a parabolic arch. The shrinking of concrete produces the same effect as a drop of temperature of -35° F; it gives, like a temperature change and in the same line of action, the horizontal reaction

$$H_s = -0.94 \frac{E \cdot \Delta s}{l} \cdot \frac{t^3}{h^2} \tag{21}$$

if Δs signifies the shortening of centre line with shrinking of concrete.

The normal stresses and their values at intrados and extrados are determined from M, N with

$$\nu_{1,2} = \frac{N}{A} \pm \frac{Me}{J} = \frac{N}{bt} \pm \frac{6M}{bt^2} \tag{22}$$

$e = \frac{t}{2} =$ distance of intrados and extrados from the centre line. Or to the primary normal stress $v_o = -\frac{p_2 r_2}{b\,t}$, constant for all the arch, are added the additional stresses produced by the horizontal force ΔH acting in the line of gravity of centre line; this force gives in each section a moment M and a thrust N, and the extreme stresses $v_{1,2}$ are then determined by equation (22). *Guidi*[10] transforms for an arch of constant thickness the formulas for stresses in the crown and abutment joint into a very simple form and adds, to facilitate calculation, numerical tables of coefficients in the equations. The stress at the crown is:

in the intrados

$$v_1 = -p\left(\frac{r}{t} - \mu_1\right) - \varepsilon E\left(\delta \frac{t}{r}\mu_1 - \frac{\delta_2 - \delta_1}{2}\right), \quad \mu_1 = \frac{1}{\vartheta''}\left(\frac{s-1}{s} + \frac{t}{6r}\right), \tag{23a}$$

in the extrados

$$v_2 = -p\left(\frac{r}{t} + \mu_2\right) + \varepsilon E\left(\delta \frac{t}{r}\mu_2 - \frac{\delta_2 - \delta_1}{2}\right), \quad \mu_2 = \frac{1}{\vartheta''}\left(\frac{s-1}{s} - \frac{t}{6r}\right); \tag{23b}$$

the stress at the abutment:

in the intrados

$$v'_1 = -p\left(\frac{r}{t} + \mu'_1\right) + \varepsilon E\left(\delta \frac{t}{r}\mu'_1 + \frac{\delta_2 - \delta_1}{2}\right), \mu'_1 = \frac{1}{\vartheta''}\left[\frac{1}{s} - \frac{r-h}{r}\left(1 + \frac{t}{6r}\right)\right] \tag{24a}$$

in the extrados

$$v'_2 = -p\left(\frac{r}{t} - \mu'_2\right) - \varepsilon E\left(\delta \frac{t}{r}\mu'_2 + \frac{\delta_2 - \delta_1}{2}\right), \mu'_2 = \frac{1}{\vartheta''}\left[\frac{1}{s} - \frac{r-h}{r}\left(1 - \frac{t}{6r}\right)\right] \tag{24b}$$

These formulas assume the temperature change to vary in a section linearly (fig. 2) with a value δ_1 at the intrados, δ_2 at the extrados and δ at the centre line.

The thickness of arch dams atteins very high values in the lower portions, in proportion to the radius of curvature and the length of arch. Thus the main condition of the usual analysis of arch, that the dimensions of sections should be small in comparison with the radius of curvature and the length of arch, is not fulfilled. For *thick arches* (great curvature) one gets the known more exact, analysis leading to the variation of normal stresses according to the law of a hyperbola, as *H. Bellet*[19] remarks; he also tries a more exact calculation of the effect of thrust and shear, but comes for normal stresses to the formula (of *Lamé*) for a thick cylindrical shell because he supposes that the angle of two adjacent sections does not change with deformation, which is true only for a thick cylindrical shell loaded with uniform radial forces.

From the assumption that plane sections remain plane, which for thick arches leads to the hyperbolic law of normal stresses, *B. F. Jakobsen*[20] derived a solution for circular arch with constant sections, loaded with uniform radial pressures. *W. Cain*[21], in his contribution to the discussion on *Jakobsen*'s paper, transformed the final equations into a better form. He obtains (fig. 5)

$$X = p_2 r_2 - H_c = \frac{p_2 r_2}{\vartheta_0} \cdot 2 \frac{i^2}{r_0^2} \sin \alpha, \tag{25}$$

$$= \left(\alpha + \frac{1}{2} \sin 2\alpha\right)\left(1 + \frac{i^2}{r_0^2}\right) - \frac{1 - \cos 2\alpha}{\alpha} + 2.88 \frac{r}{r_0} \cdot \frac{i^2}{r_0^2}\left(\alpha - \frac{1}{2} \sin 2\alpha\right), \tag{25a}$$

if r_0 signifies the radius of the neutral line, which differs here from the centre line; the difference is

$$r - r_0 = c = r - \frac{t}{\log \text{nat.} \left(\frac{r_2}{r_1}\right)} \tag{26}$$

About any point M_0 of the neutral line, given with the angle φ of the radius OM_0 with the axis of symmetry OC, the external forces at one side of the section OM_0 give a moment

$$M = - X r_0 \left(\frac{\sin \alpha}{\alpha} - \cos \varphi\right); \tag{27}$$

it is the moment about the point M_0 of the force X, acting to the right, substituting the right half of arch, at a distance of $r_0 \frac{\sin \alpha}{\alpha}$ from the centre O, viz. in the centre of gravity of the neutral line. In the section given by the angle φ one

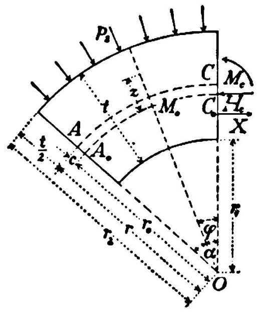

Fig. 5.

has also the thrust according to equation (7) $N = X \cos \varphi - p_2 r_2$ and the shear according to (8) $T = X \sin \varphi$; to the thrust $- p_2 r_2$ uniformly distributed in the section there comes the force X formerly mentioned.

Moment M and thrust N gives at a distance z from the neutral line the normal stress

$$v = \frac{N r_0}{(r_0 + z)\, t} - \frac{M}{J} \cdot \frac{r_0 z}{r_0 + z}; \tag{28}$$

N and v are positive as tensions, moment M is positive when acting clockwise for forces on the left of the section, and z is positive for the outer side of the neutral line. From (28) one gets the stresses at the extrados with $z = \frac{t}{2} + c$, $r_0 + z = r_2$ and at the intrados with $z = -\left(\frac{t}{2} - c\right)$, $r_0 + z = r_1$.

The water pressure produces a deflection of the crown (positive in the direction to the center O)

$$\eta = \omega_0 \cdot \frac{p_2 r_2 r_0}{E t}, \tag{29}$$

where*

$$\omega_0 = \frac{1}{\vartheta_0}(1 - \cos\alpha)\left[(\alpha - \sin\alpha)\left(1 + \frac{i^2}{r_0^2}\right) + 2.88\frac{r}{r_0}\cdot\frac{i^2}{r_0^2}(\alpha + \sin\alpha)\right]. \tag{29a}$$

With respect to formulas for thin arches the equations for thick arches give a lesser tension and a greater compression; the effect of great curvature of arch is therefore advantageous.

A constant temperature change gives a horizontal reaction

$$H_t = \delta\,\varepsilon\,E\,t\frac{i^2}{r_0^2}\cdot\frac{2\sin\alpha}{\vartheta_0}, \tag{30}$$

acting in the line of gravity of the neutral line. The deflection of the crown from temperature change is

$$\eta_t = -\,\omega_0\cdot\delta\,\varepsilon\,r_0. \tag{31}$$

To facilitate the computation with *Cain*'s formulas, *F. H. Fowler*[22] elaborated for thin and thick arches *diagrams* for resulting normal stresses at intrados and extrados of the crown and abutment joint. The numerical results show that the shear can be neglected for $t/r = 0.02$ to 0.06.

The equations for thick arches give good results if the thickness of the arch is not too great. For too great dimensions, such as sometimes appear in the lower parts of arch dams, even this analysis is inexact. A correct calculation of stresses should be based on the mathematical theory of elasticity; *R. Chambaud*[23] showed that it gives in this case very good results. He proceeds from the mathematical theory of elasticity and introduces no other hypothesis than *Hooke*'s law. *Chambaud* gives the solution for an arch of rectangular section; it can be applied to all thick arches (arch dams, tunnels and underground conduits), further to thick cylindrical shells. This theory naturally gives complicated formulas, but numerous diagrams allow a quick and simple application. The results correspond very well to all surface conditions, except a small extent at the abutments; they can be adapted for any distribution of external forces on the intrados and extrados, and for any distribution of internal strains, therefore for various shrinkings in several places (caused for instance by the method of construction) or for irregular temperature changes. The solution is especially valuable, because it usually gives much more favourable results than the theory of thick arches previously mentioned. The usual theory of thick arches (and still more the usual theory of thin arches based on linear distribution of stresses in sections) leads as a rule to greater tensions on the intrados at the crown and especially on the extrados at the abutments, where this theory indicates the weakest point of dam. Great tensions would cause cracks in an arch without reinforcing and the consequence would be that the uninjured masonry would form a new arch able to resist safely the external forces; this was at first observed by *J. Résal*[1] (he supposed the "acting" arch parabolic), afterwards by *M. Malterre*[24] (with

* There is an error in Cain's paper (Transact A.S.C.E., vol. 90, p. 541, form. 109), as clearly shows comparison with the preceding equation.

the "acting" arch circular, of constant and variable thickness) and *L. J. Mensch*[25]. The exact calculation by the theory of *Chambaud* shows that the actual stresses are much more favourable; especially the tensions on the extrados disappear (which is particulary important for the impermeability of the dam), the tensions on the intrados are limited at most to a small portion at the crown. The exact solution gives on the whole few differences with respect to the usual theory of thick arches as regards the effect of bending moments; a considerable difference appears in the effect of thrust which outweighs the effect of bending moments in thick arches, if exactly calculated. The differences in the stresses mainly concern the neighbourhood of intrados. Moreover, the exact theory makes due allowance for the shearing force. The usual theory of thick arches does not give good results for too great thickness, because it is based on assumptions which are not correct: it neglects the normal stresses in radial direction and determines the normal stresses in sections, as though plane sections would remain plane after deformation. Especially the last hypothesis is not right for curved bars (arches), because there the determination of the effect of normal and shearing stresses cannot be divided as for straight bars. The exact theory gives for normal stresses (in the direction of radius ν_1, of tangent to the arch ν_2 and in the direction of the axis of intrados ν_3) and for shearing stresses τ (perpendicular to the axis in the radial section and in the cylindrical section) altogether curves;

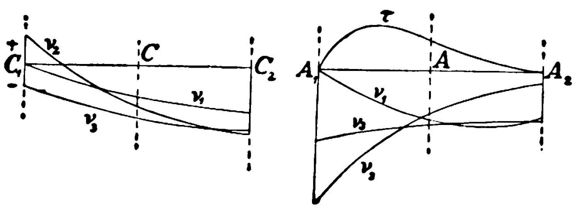

Fig. 6.

fig. 6 shows these curves for the crown section $C_1\,C_2$ and for the abutment $A_1\,A_2$ of an arch with radius $r = t = C_1\,C_2 = A_1\,A_2$. Chambaud made the analysis for an arch with external forces and stresses symmetrical to the plane of centre lines. The application for other cases naturally gives only approximate results.

The analysis of the arch dam as a system of horizontal arches independently withstanding the water pressure and the effects of temperature changes, shrinking and swelling of concrete, can be very good if for instance when constructing in layers the connection of layers in a vertical direction is destroyed; this can be seen at sudden breaks of deflection lines of vertical sections[15]. This analysis would be exact if the dam were actually divided into independent horizontal arches with horizontal contraction joints, filled with asphalt and bent copper sheets to obtain impermeability, as planned *by A. Peña Boeuf*[26]. Otherwise this analysis is only approximate.

Analysis of Arch Dam as a System of Horizontal Arches and Vertical Cantilevers.

In reality the horizontal arches hang together in a vertical direction and cannot deform quite independently; this causes a reciprocal action of horizontal arches in a vertical direction. A more exact analysis of arch dams considers the dam as divided by horizontal sections into horizontal arches and by vertical radial sections into vertical cantilevers. Between these two systems are distributed the external forces. The conditions of this distribution are given by the deformation of the dam, which must be equal at every point for the two systems. If we were to consider all the components of deformation at each point (three components of displacement in three perpendicular axes and three components of turning about these three axes), we should obtain an exact solution. Since this method of calculation is almost impossible practically, it is simplified by disregarding all turnings and the respective torsional stresses, by disregarding also the tangential component of horizontal displacement and the respective shearing stress. Moreover one can also disregard the vertical component of displacement, if one considers the dam after deformation by the weight of masonry is completed. There remains only the horizontal component of displacement perpendicular to the centre line of horizontal arch (radial displacement), and in consequence of this only one condition for each point where the centre line of supposed horizontal arch and the axis of vertical cantilever cross. Thus we substitute for the dam a system of vertical cantilevers and horizontal arches which simply (without restraining) support one another[27]. The torsional stresses, omitted by this method, in reality diminish a little the bending stresses and increase security.

An exact analysis by this method would be difficult, because the displacement of any point of the cantilever (or arch) depends on all loads acting on the cantilever (arch). The conditions of equal displacements of horizontal arches and vertical cantilevers in all points therefore give equations, each of which contains a great number of unknown quantities.

A. H. Woodard[28] simplifies the calculation regarding the deformation of the dam only in the vertical section through crowns of arches (where the dam is highest); he supposes the arch under simple compression, determines the deflection of the crown as for an arch with two hinges and takes the distribution of water pressure between the system of horizontal arches and vertical cantilevers, computed from the crown section, uniform along the arches. *R. Schirreffs*[29] endeavoured to improve the analysis by calculating the deflection of the arch crown as for a hingeless arch, otherwise using the same method of analysis; but he disregarded the effect of thrust and his formula is too complicated and incorrect, as *W. Cain*[11] showed. *H. Bellet*[19] determines the distribution of pressure between arches and cantilevers from a wrong supposition that the strain of centre line of arch at any point equals zero.

H. Ritter[9] in a numerical example (in 1913) proceeded approximately, supposing on each horizontal arch a uniform radial loading and determining its value by equating the deflection of arch crown and vertical cantilever in the middle vertical section. Analogically *L. R. Jorgensen*[30] examines only the middle vertical section, but computes the distribution of pressure only with a rough approximation; *L. J. Mensch*[31] uses for calculation of pressure distribution on cantilever and horizontal arches the unsuitable condition of equality of internal works. *J. Résal*[1] also considers only the middle vertical section.

H. Ritter[32] indicated the principle of a more exact calculation of load distribution on vertical cantilevers and horizontal arches thus: The deflection at any point M of the vertical cantilever A B (fig. 7) can be computed from its influence line (viz. deflection line of the cantilever A B loated with P = 1 in the point M): it has a value

$$\eta_m = \Sigma P'_n \eta_{nm} \tag{32}$$

if P'_n designates a load acting at the point N on the cantilever. This deflection equals the deflection of the horizontal arch at the same point with a load $P''_n = P_n - P'_n$; P_n is the total load at the point N. We thus obtain as many equations as we take horizontal elements, supposing that the loading of horizontal arches is uniformly distributed and that there is in consequence only one value P''_n for each horizontal arch to compute from these equations. In this manner we could proceed for any vertical section of the dam and we would find for different vertical sections various loads on the horizontal arches; the loading of these arches is therefore not uniform.

A. Stucky[14] is the first to consider actually (in the analysis of the dam on the river Jogue, made with the cooperation of prof. *A. Rohn*) all vertical cantilevers and horizontal arches (both of variable sections) and to take account not only of the different spans and rises of arches, but also of the different heights of vertical sections, which have an essential influence on their stiffness and therefore on the distribution of water pressure on vertical cantilevers and horizontal

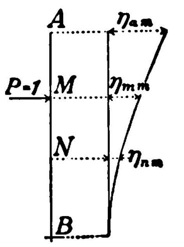

Fig 7.

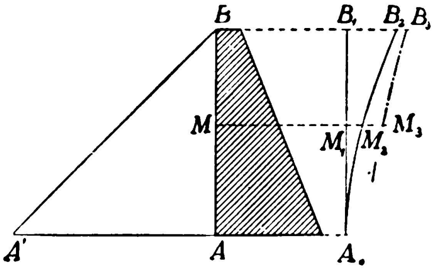

Fig. 8.

arches. The evolution of the resultant equations can be facilitated by solving separately the system of equations concerning each vertical cantilever (considering thereby only the loads on this cantilever). The approximate values calculated can thus be improved from original equations by iterative calculation. Since the exact fulfilment of suppositions of the analysis cannot be warranted for masonry dams with respect to the execution and the material used, each analysis of dam is to be considered as approximate; therefore the results of the first approximate solution are often sufficient. The results can be checked by calculating the deflections of vertical cantilevers and horizontal arches for the determined distribution of loads; it suffices if both deflections at the same point do not differ by more than 10 %.

A practical *trial method* was given by *F. A. Noetzli*[17] and completed by *W. Cain*[33]. It is first ascertained whether the horizontal arches act on the whole. To this end we determine the deflection line $A_1M_2B_2$ (fig. 8) of the vertical cantilever between two vertical radial sections in the middle of the dam, for the whole water pressure A A'B. In addition, we determine the deflections of horizontal arches, supposing them to bear the full water pressure. If the deflections of the vertical cantilevers are throughout smaller than the deflections of the arches (line $B_3 M_3$), the cantilevers bear all the load; the arches could be stressed only if the temperature decreases and diminishes their deflection. This case occurs if the thickness of the dam is calculated by neglecting the influence of arches (as for a straight dam).

If the thickness of the dam is smaller, part of the water pressure is borne by vertical cantilevers, part acts on horizontal arches. The vertical cantilever bears at the base the full water pressure, because its deflection is very small there (smaller than the deflect on of arch with full water pressure). From the base to the top of dam the load of the arches increases approximately according to a straight line AB′ (fig. 9); in the upper part of dam the arches, being stiff enough, hinder the deflection of cantilever (deflect less than the cantilever and support it), therefore act on the cantilever with reactions opposed to water pressure. From the load diagram of water pressure AA′B the arches bear the part AB′B, the vertical cantilever the part AA′BB′ (AA′C′ is positive, C′B′B negative). We consider the highest vertical section and assume on the arches an approximately uniform loading. For the load diagram of the vertical cantilever it is easy to obtain (best by calculating) the bending moments and to determine the deflection curve of the cantilever as a funicular polygon to the loading diagram with ordinates $M \frac{J_o}{J}$; J_o is a constant moment of inertia, J the moment of inertia of the section. At a chosen point C all the load is to be borne by the arch. We

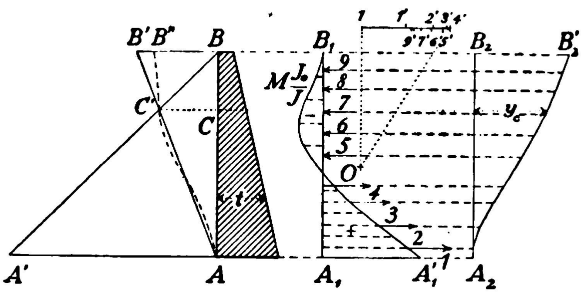

Fig. 9.

determine at C the deflection of the arch crown for full water pressure. If the cantilever has at C a greater deflection y_c than the arch, it is necessary to choose the point C lower and repeat the calculation. The exact position of C is determined with a linear interpolation between the two points C_1, C_2 formerly chosen (after fig. 10, where $C_1 C'_1$, $C_2 C'_2$ are arch deflections and $C_1 C''_1$, $C_2 C''_2$ cantilever deflections and the arch deflections throughout the height of dam. Usually there will not be complete accordance. To obtain equal deflections not only at C, but also on the top, we must change the load diagram for horizontal arches by substituting the straight line C′B″ for C′B′; the arches then support diagram AC′B″B (AC′A′ is positive, C′B″B negative); the vertical cantilever supports diagram AC′B″BA′. We change the point B″ until we have at C and B equal deflection of arch and cantilever. At other points the deflections need not be the same, because the broken line AC′B″ should be actually a curve. We determine it by assuming on the arches a smaller (greater) load, where the calculated arch deflection is greater (smaller) than the cantilever deflection.

The water pressure produces in cantilevers the greatest stresses in the lowest joint, where greater tensions can occur on the up-stream side. If there is no reinforcement, horizontal cracks on the up-stream face at the base of dam can appear. In this case vertical cantilever does not act as a beam perfectly fixed, but only as a beam partially fixed or hinged at the base. We can then find the right solution by trail, choosing the tangent to the deflection line at the base of the cantilever, otherwise calculating as formerly indicated and ascertaining whether the deflections of the cantilever coincide throughout with the arch deflections.

R. Chambaud[23] also indicates a method of finding the division of loading on horizontal arches and vertical cantilevers. He proceeds from any (approximate) law for the part of water pressure carried by the arches, supposes in each horizontal arch an approximately uniform loading and computes the deflections of arch crowns and the deflections of cantilevers under the load carried by them. For the second computation he introduces half the sum of these deflections, determines from it the division of loading between arches and cantilever and repeats the computation. Thus he can approach the exact values. He also considers approximately the normal stresses in the vertical direction (of the axis of arch) with their average value.

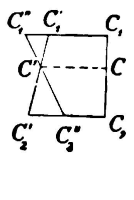

Fig. 10.

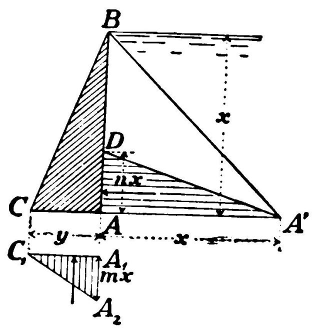

Fig. 11.

A. Rohn[34] recommends for the first calculation this *approximate method:* The vertical cantilever is supposed to carry from the load diagram AA'B (fig. 11) of the whole water pressure the triangular part AA'D with the base AA' $=$ x $=$ height of dam, and $\overline{AD} = n \cdot x$, where $n = \frac{1}{5}$ to $\frac{1}{2}$ for $\frac{b}{h} = 1.1$ to 1.8; b is the length of dam at the top h its height. The rest of water pressure acts on horizontal arches. Besides he always recommends consideration of the *uplift* with a triangular load diagram $A_1C_1A_2$ (as for straight dams), where $\overline{A_1A_2} = m \cdot x$ for $m \leqq 1$; in the upper part of dam $m = 0.8$ suffices. For a triangular section of dam the necessary thickness at the base is

$$y = n \cdot x \sqrt{\frac{1}{\gamma - m}}; \tag{33}$$

$\gamma =$ weight of masonry in proportion to the weight of the same volume of water. For $m = 1$, $n = \frac{1}{4}$, $\gamma = 2.3$ the result would be $y = 0.22\, x$.

The uniform distribution of radial pressures on horizontal arches assumed in the majority of approximate methods of analysis, is not sufficiently exact.

The division of water pressure on horizontal arches and vertical cantilevers depends very essentialy on the form of the cross-section of the valley. It is therefore necessary for an exact analysis to consider not only one (the highest) cantilever, but a greater number of vertical cantilevers and horizontal arches: it was thus that *A. Stucky*[14] proceeded. Another trial method was given by *C. H. Howell* and *A. C. Jaquith*[35], who choose a note uniform loading of arches, determine for this loading the deflections of arches and for the remaining loading the deflections of cantilevers and vary successive the loading of arches until they get at all points practically equal deflections of arches and cantilevers. It is necessary to make more trials in order to obtain a satisfactory coincidence. From the resultant loading the stresses in arches and cantilevers can be computed. In their analysis *Howell* and *Jacquith* omit the non-active extended parts of arches and cantilevers and limit the final calculation of the dam (without reinforcement) only to the parts working in compression: they always have arches of variable section which they calculate omitting the influence of shear.

Comparison of several cases showed that the analysis of arch dam as a system of independent horizontal arches is not exact and that it requires more masonry, especially for calculating the arches in a rough approximation as thin cylindrical shells, as was formerly the custom. The influence of vertical cantilevers should

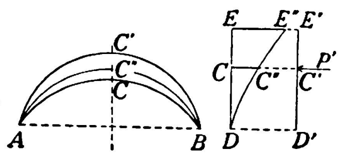

Fig. 12.

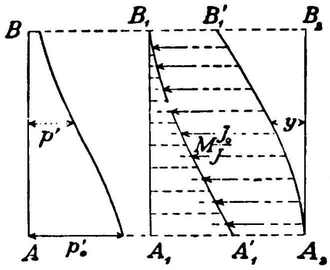

Fig. 13.

not be omitted, as it always appears and alters the loading and condition of stress of horizontal arches. The last method of analysis is available for any profile of the dam site, also for unsymmetrical profile.

The *influence of temperature changes,* which can produce greater stresses than water pressure can be computed in the same way as the latter. It can be even substituted (*Ritter*[9]) by a water pressure which gives the same deflections of horizontal arches as temperature change; this equivalent water pressure is to be divided over the system of vertical cantilevers and horizontal arches analogically as a real water pressure.

A trial solution of the influence of temperature change was given by *F. A. Noetzli*[17] and improved by *W. Cain*[21]. We again suppose the dam to be divided into vertical cantilevers and horizontal arches. The centre line of arch ACB (fig. 12), fixed at the ends and otherwise free, would deform by temperature change in AC′B; the displacement of the crown would be, according to the formula (31) for thick arches

$$\eta_t = \overline{CC'} = -\omega_o \cdot \delta\, \varepsilon\, r_o.$$

This displacement is hindered by the reactions p′ of vertical cantilevers DCE;

supposing them to be constant on the length of each arch, we have after (29) the displacement

$$\eta' = \overline{C''C'} = \omega_0 \cdot \frac{p' r_2 r_0}{E t}$$

The resulting diplacement is

$$y = \overline{C\,C''} = \omega_0 \cdot r_0 \left(\frac{p' r_2}{E t} - \delta\,\varepsilon\right) \tag{34}$$

The resulting deflection curve of vertical cantilever is D C″ E″. The loading p′ will be determined by analysis of dam. In the base of dam there is

$$y = \omega_0\, r_0 \left(\frac{p'_0 r_2}{E t} - \delta\,\varepsilon\right) = 0$$

therefore

$$p'_0 = \frac{\delta\,\varepsilon\, E\, t}{r_2}$$

We then choose at the crown a slight specific pressure (fig. 13) and in the vertical section a curve for distribution of pressures p′ (in the first attempt we can choose a straight line). For this loading we determine for the vertical cantilever the bending moments M and the values $M\,\frac{J_0}{J}$; J is the moment of inertia of the section of cantilever, J_0 a constant moment of inertia. The line $M\,\frac{J_0}{J}$ gives the loading diagram for deflection curve as a funicular line. The analysis is correct if the deflections y of vertical cantilever coincide with the deflections of arches computed from equation (34); the loading of arches is given by p′ in the opposite direction as for vertical cantilevers. If there is no coincidence, it is necessary to correct the computation by altering the loading curve for p′.

The diminution of temperature can be combined with the *shrinking of concrete;* if ε′ is the shrinking for unit of length, the resulting deflection of arch crown is

$$y = \overline{C\,C'} = \omega_0\, r_0 \left(\varepsilon' - \delta\,\varepsilon - \frac{p' r_2}{E t}\right); \tag{34a}$$

the temperature change δ is here negative, the reaction p′ of vertical cantilevers (in the last equation positive) acts from the centre of arch. The shrinking of concrete has the same influence as temperature change (diminution), which would cause a shortening equal to that caused by shrinking.

An increase of temperature causes a deflection of dam up-stream for empty reservoir; vertical sections also bend up-stream, which in vertical cantilevers produces tensions in the down-stream surface of the lower part of dam. In the arches, on the contrary, tensions are produced at the crown on the up-stream face; there cracks can develop if there is no reinforcement. With reservoir full and diminution of temperature the dam moves down-stream; there may be a tension in the cantilever in the lower part on the up-stream face in the arches tensions at crown on the down-stream face. For all tensions there should be adequate reinforcement; otherwise vertical cracks could occur gradually in the arch crowns on both sides, which would affect the stability of dam very unfavourably. It the distribution of loading on vertical cantilevers and horizontal arches is neglected (only the resistance of arches is considered), wrong construction can easily cause horizontal cracks, as the results of measurements on some dams appear to show [17].

As concerns the amount of temperature changes, *F. A. Noetzli* [17] therefore recommends for higher dams at the base thicker at the top greatest temperature change ($\pm 25^0$ F) be considered, at the base no change and between them linearly variable changes; for exact calculation we have not as yet enough results of actual measurements. At *Arrow-Rock* dam [36] the yearly change of temperature at the top was found to be 27^0 F, at the base only 6.5^0 F. There can also be several combinations of temperature changes on the up-stream and down-stream face; it is particularly necessary to consider for an empty reservoir the same largest drop of temperature on the up-stream and down-stream face, and for a full reservoir different diminutions of temperature on the up-stream face (to the lowest temperature of water) and on the down-stream face (to the lowest temperature of air).

In thicker dams the temperature changes do not penetrate the whole dam equally; a closer examination of it is given by *A. Stucky* [14]. *G. Ippolito* [13] examines in detail the masonry and derives simple formulas for distribution of temperature in the latter; they can be used for any masonry structure to determine daily and yearly changes of temperature. The same author also examines the influence of temperature rise on the hardening of concrete and gives results of temperature measurements on several dams; these are but few and do not permit safe conclusions to be arrived at. The calculations usually give too large stresses from temperature changes if one considers the temperature change constant or linearly variable through the thickness of dam, which does not correspond to reality. The deformations caused by temperature changes can also have a favourable influence on the stresses if there is unelastic yielding in the abutments or in the interior of dam.

A simple formula for the penetration of temperature changes in the interior of thick masonry, derived from American measurements, is given by *H. Ritter* [9]:

$$\delta = \frac{\delta_1}{3\sqrt[3]{x}} \tag{35a}$$

where δ is the temperature change in the masonry at a distance x from the surface. δ_1 the temperature change of air. *G. Paaswell* [37] develops for this case the formula

$$\delta = \delta_1\, e^{-kx} \cos kx \tag{35b}$$

k is a constant dependent on material and time: for concrete and the period of one day $k = 0.079$, for concrete and the period of one year $k = 0.00413$.

Too great influence of temperature changes and shrinking of concrete can be eliminated by *contraction joints.* For a dam calculated as gravity dam these joints are statically inoffensive. For an arch dam too many contraction joints are unfavourable with respect to stability.

Analysis of Arch Dam as an Elastic Shell.

An arch dam is in reality an elastic shell, free on top and supported or fixed on other parts of its circumference to the sides and bottom of the valley. But the analysis of an arch dam as elastic shell is very difficult. It is necessary to start from equilibrium and deformation of an infinitesimal element (as in the analysis of flat plates) and to satisfy the boundary conditions at the abutments and at the top of dam. The idea of this analysis was formulated generally by *G. Pigeaud*[3].

B. A. Smith[38] was the first to attempt to calculate an arch dam as an elastic shell. He simplified his analysis, considering only the highest part of dam and assuming in the horizontal direction throughout the dam the same conditions as for the highest section; he eliminated in this way the variability in horizontal direction (dependence on central angle φ). He considers the boundary conditions only for the top and base of the vertical section; this is in reality in accordance with the analysis of a vertical cylindrical shell of a reservoir. The connection of elements in a horizontal direction is considered in stresses, but not in deformation; it is only shown with a rough approximation that for central angles, smaller than 120^0, the deflection of the crown of horizontal arch can be computed as for a full circle, substituting the real modulus of elasticity E_0 for the arch with $\frac{2}{3}\,E_0$. *Smith* also considers the shearing forces in horizontal planes and from equilibrium conditions of forces acting on the element $t \cdot ds \cdot dy$ (between two horizontal planes, two vertical radial planes and the up-stream and down-stream face of dam), from deformation of vertical cantilever by bending moment and of horizontal arch by thrust (the bending moments in arches are neglected) develops the fundamental equation

$$\frac{d^2}{dy^2}\left(C_1\,\frac{d^2 z}{dy^2}\right) + \frac{E_0}{r^2_2}\,t\,z = p\,; \tag{36}$$

r_2 is the radius of up-stream face (fig. 14), p the external (water) pressure uniformly distributed along the horizontal arch, t = thickness of dam, $C_1 = E_1 J = \frac{1}{12}\,E_1 t^3$ is the flexural rigidity (for a vertical element of horizontal length = unit of length), E_1 modulus of elasticity for vertical cantilever (can be different from E_0 for horizontal arch, if there is another reinforcement), z = radial deformation (deflection and y = depth measured from water surface (at the top of dam) in the direction of vertical axis of dam surfaces. The analysis erroneously considers the vertical cantilever as an independent element, without connection with other elements; therefore *Poisson*'s ratio escapes from the resulting equations.

Smith gives the analysis for a dam of constant thickness and for a dam of trapezoidal vertical section. In the first case the solution is similar to the known solution for cylindrical shell of reservoir; only *Poisson's* ratio is not in the results. For a thickness linearly variable the solution contains series in the form of special *Michell's functions;* the author's paper gives numerical tables of these functions to facilitate the calculations and derives the connection with complex *Bessel's functions.*

W. Cain[33] showed in a numerical example that the methods of *Smith* and *Noetzli* give absolutely identical results, though *Noetzli* neglected the shearing forces; these forces can therefore be neglected. The accordance of both methods is natural enough, because their basis is in reality the same: both consider the vertical cantilever at the middle of dam and neglect the variation of values in a horizontal direction. The only difference is that *Smith* integrates a differential equation, also uses infinitesimal elements, whereas *Noetzli* considers finite elements. But this has no essential influence on the results if the number of elements of the vertical section is not too small.

G. Paaswell[37] derives from fundamental relations for deformation and from the energy expended on deformation the general flexural equation for an elastic shell

$$p = \frac{E\,J}{1-\mu^2}\left(\frac{\partial^4 z}{\partial y^4} + \frac{2}{r^2}\cdot\frac{\partial^4 z}{\partial y^2\,\partial\varphi^2} + \frac{1}{r^4}\cdot\frac{\partial^4 z}{\partial\varphi^4} + \frac{2}{r^2}\cdot\frac{\partial^2 z}{\partial y^2} + \frac{2}{r^4}\cdot\frac{\partial^2 z}{\partial\varphi^2} + \frac{z}{r^4}\right); \qquad (37)$$

z = deflection of shell, y = vertical distance from water surface (fig. 15), φ = angle measured in a horizontal plane from the plane of symmetry, r = radius of middle cylindrical surface, p = radial external pressure (water pressure) acting on the shell, and μ = *Poisson*'s ratio. For $r = \infty$ (and $r \cdot d\varphi = dx$) the equation (37) transforms itself into the fundamental equation for

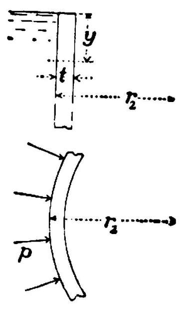

Fig. 14.

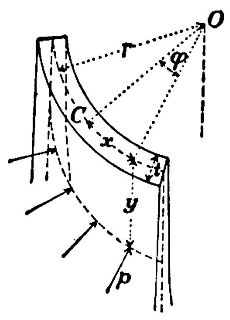

Fig. 15.

flat plates. The author does not determine the general integral of equation (37); he only gives a particular solution and derives from it the relation of bending moments in cantilever and arch. He shows that the bending moments in the cantilever are much greater at the base of dam than the moments in the arch, and that the arch moments change their sign in the lower part of dam.

In the excellent paper "Report on Arch Dam Investigation, Vol. I"[2] *H. M. Westergaard* deals theoretically with the analysis of arch dam as an elastic shell; he considers in radial and horizontal sections thrusts and two components of shear (in a radial and perpendicular direction), further bending moments for vertical sections and horizontal arches, and twisting moments; the distribution of stresses is supposed to be normal as for flat plates; and shearing stresses linearly variable through the thickness of dam, assuming a dam of small thickness. The author derives from the equilibrium and deformation of an element between two horizontal planes with a distance dy, two radial planes with a distance dx in the middle circle of radius r, and the down-stream and up-stream face of dam the equation of flexure

$$\frac{\partial^4 z}{\partial x^4} + 2\frac{\partial^4 z}{\partial x^2 \partial y^2} + \frac{\partial^4 z}{\partial y^4} + \frac{1}{r^2}\cdot\frac{\partial^2 z}{\partial x^2} + \frac{\mu}{r^2}\cdot\frac{\partial^2 z}{\partial y^2} + K\left(\frac{\partial^3 z}{\partial y^3} + \frac{\partial^3 z}{\partial x^2 \partial y} + \frac{\mu}{r^2}\cdot\frac{\partial z}{\partial y}\right)$$
$$+ k\left(\frac{\partial^2 z}{\partial y^2} + \frac{\mu\,\partial^2 z}{\partial x^2} + \frac{\mu}{r^2}\cdot z\right) - \frac{1}{N}\left(p - \frac{P_x}{r} + P_y r'' + \gamma t r'\right) = 0 \qquad (38)$$

and the differential equation of central forces

$$\frac{\partial^4 F}{\partial x^4} + 2\frac{\partial^4 F}{\partial x^2 \partial y^2} + \frac{\partial^4 F}{\partial y^4} - \frac{E t}{r}\cdot\frac{\partial^2 z}{\partial y^2} = 0 \qquad (39)$$

In these equations z is the deflection of dam, r = the radius of cylinder of reference (fig. 15), x = distance measured on this cylinder in the horizontal direction from the vertical plane of symmetry (going through axis of symmetry O C at the top, y = the vertical distance from the top of dam, t = the thickness of dam: further there is $r' = \frac{d\,r_y}{dy}$, $r'' = \frac{d^2 r_y}{dy^2}$, where r_y = the radius of the middle surface (dependent only on y), E = the modulus of elasticity of masonry, μ = *Poisson*'s ratio (for concrete $\mu = 0.15$), $N = \frac{E t^3}{12(1-\mu^2)}$ = the measure of stiffness of the dam in flexure,

$$K = \frac{2N'}{N},\quad k = \frac{N''}{N},\quad N' = \frac{dN}{dy},\quad N'' = \frac{d^2 N}{d y^2},$$

p = the water pressure per unit of area of the cylinder with radius r, γ = the weight of masonry per unit of volume, P_x = the horizontal thrust per unit of length of vertical radial section, P_y = the vertical thrust per unit of length of horizontal section. Finally, F denotes the stress function determining the forces P_x, P_y, P_{xy} by means of equations

$$\frac{\partial^2 F}{\partial y^2} = P_x,\quad \frac{\partial^2 F}{\partial x^2} + \gamma\int_0^y t\,dy = P_y,\quad P_{xy} = -\frac{\partial^2 F}{\partial x\,\partial y}; \qquad (40)$$

P_{xy} is the vertical central shear per unit of length of radial section.

In the same paper *W. Slater* derives from the differential equation for a flat plate a simpler differential equation for flexure of arch dam

$$\frac{\partial^4 z}{\partial x^4} + 2\frac{\partial^4 z}{\partial x^2 \partial y^2} + \frac{\partial^4 z}{\partial y^4} + \frac{1}{r^2}\cdot\frac{\partial^2 z}{\partial x^2} - \frac{A}{J r}\left(\lambda_x + \mu\,\lambda_y\right) = p\cdot\frac{1-\mu^2}{E J} \qquad (41)$$

A = area of element in the vertical radial section, J = its moment of inertia; λ_x and λ_y are the horizontal and vertical strains in the direction x and y.

For an exact analysis of dam it would be necessary to solve the differential equations (38), (39), considering the boundary conditions on the top, where the dam is free, and at the abutments. In calculating the dam, we can according to *Fred. Vogt*[39] consider the deformation of bed-rock. *Fred Vogt*[40] determined the formulas and calculated numerically the influence of the *yielding of the rock foundation* in an arch dam. He came to the result that the yielding of the rock foundation can be very approximately computed by extending the dam to the imagined fixed foundation at a distance of 0.45 t from the abutment. The yielding of the foundations naturally alters the stresses and the deformations of an arch dam. For small thicknesses this yielding has no substantial influence. For great thickness (in the lower parts of high arch dams) the yielding of foundations diminishes the bending moment at the abutment of arch, thus diminishing the tension at the extrados of arch; on the other hand, the bending moment at the crown of arch and the tension at intrados increases. Stresses from temperature change, shrinkage and swelling of concrete become regularly smaller in consequence of yielding, the deflection of crown becomes considerably greater (up to a twofold value).

The solution of fundamental differential equations (38), (39) for an arch dam is very complicated and difficult. For ordinary practical cases this method of analysis is too laborious.

Shape of Arch Dams.

At first arch dams were generally constructed with vertical up-stream face and a constant radius of curvature in all horizontal sections or even with a radius increasing towards the bottom. Such a form is convenient for a constant width in all horizontal sections, therefore for an arch between vertical piers, although even there it is better to use a smaller radius with a greater thickness in the lower parts in order to obtain more flexible arches. If the dam is in a valley whose width diminishes downwards, the arches in the lower parts are very flat. In calculating the dam as a system of vertical cantilevers and horizontal arches (this is also assumed in the following cases), we get a relatively small portion of water pressure on the arches, the greater part of it being carried by the vertical cantilevers; because of bending in vertical cantilevers much masonry is required. It is therefore better to diminish the radius of curvature from top to bottom; this transfers the greater part of the load on the arches, where the stresses are more uniform and the strength of concrete is better utilised (*Stucky*[14]). *L. R. Jorgensen*[30] therefore projects dams with constant central angle in all horizontal planes; these dams were often built in America in great dimensions. But the constancy of the central angle is not necessary and it cannot in practice be exactly attainded.

The idea of a constant central angle was definitely expressed as early as 1879 by *Pelletreau,*[5] who was also the first to determine the best value of the central angle if the effect of thrust be disregarded with the approximate value of 134^0, which gives the least volume of arch. If the effect of thurst is considered, the best central angle, according to *Ritter*[9] lies between 120^0 and 180^0; in this interval the volume of the necessary concrete varies but little. Tensile stresses in a thin arch with radial pressures uniformly distributed are obviated, if for a constant thickness the central angle is greater than 158^0; for a smaller central angle tensions on the extrados at the abutments can be excluded by reinforcing the arch at the abutments, if the central angle is greater than 115^0. As for the effect of temperature changes, *Ritter*[9] shows that a semicircular arch is the best.

The conditions of greatest possible economy were examined in detail by *Ippolito*[13] on the basis of the analysis of an elastic arch of relatively small thickness and calculating the arch dam as a system of independent horizontal arches. He shows that for an arch of constant thickness the best central angle is between 133^0 and 180^0 and that it depends on the depth of water; at a depth in meters equal numerically to the allowable stress k in kg/cm^2, the best angle is approximately 180^0. He further determines for an arch of constant width (i. e. for an arch between vertical piers) the best central angle for which the volume of the whole dam is smallest. For an arch dam in a valley whose breadth varies with the elevation, he construed graphical tables which permit the determination, for a given central angle at the top of dam, of the volume of arch rings in various elevations (for a constant radius of middle surface or of up-stream face), the volume of the whole dam and, by means of comparison of results for different central angles, the best central angle at the top leading to least volume of the whole dam.

For a uniform radial pressure the best form of central line is a circle, which is also convenient for construction. In reality the pressure on the arches is not uniformly distributed, because in consequence of the different resistance of vertical cantilevers differing in height (when calculating the dam as a system of horizontal arches and vertical cantilevers) the arches have to bear at various points different portions of the whole water pressure. It would naturally be possible to adjust the form of dam to this, choosing for the central line in each horizontal section the funicular line for the calculated loads on the arch (*Stucky*[14]). The possible saving of concrete would probably be outweighed by disadvantages regarding construction for which the best are circular arches also permitting the simplest calculations[41]. Moreover, the determination of the distribution of the pressure on the arch is complicated enough and cannot be performed exactly; if the form of arch is adjusted to the calculated division of pressure, it may be that the real division of load is different and does not suit the determined form of arch, so that the real stresses may exceed the calculated extreme values.

The triangular vertical section, suitable for straight or slightly curved dams wherein the arch action is disregarded, is not convenient for arch dams. With respect to stresses, it is convenient to make the dam very thin and to strengthen it towards the abutments and the sides and bottom of the valley (*Stucky*[14]), especially in case of dams of moderate height (up to 30 m), where one cannot attain the strength of material and where in the lower parts a considerable loading falls on the vertical cantilevers. A multiple statical indetermination in the distribution of loading on the system of vertical cantilevers and horizontal arches makes the arch dam very sensible to changes of dimensions; with convenient changes of thickness one can always improve the utilisation of strength of material or diminish the volume of masonry, for these changes can essentially alter the flexibility of the horizontal arches and vertical cantilevers and therefore the distribution of external forces on both systems, as *Howell* and *Jaquith*[35] have shown.

Even for very high arch dams strengthening at the abutments is advantageous, in larger valleys also a vertical up-stream face, giving a greater rigidity to vertical cantilevers[35]. The effect of uplift is much less dangerous to arch dams than to straight dams, since the abutments at the sides themselves prevent the overturning of dam. But it is necessary to consider the uplift especially where the greatest part of the loading acts on the vertical cantilevers.

Special care is necessary in determining the dimensions of the dam in places where the breadth of valley changes rapidly. Also the width of arches in adjacent horizontal elements is there very different and these arches would have very different deformations. To avoid too great shearing stresses, it is convenient to establish there massive abutments for arch dams, giving with the sides and bottom of the valley below them, a more regular form the circumference of the arch dam, and permitting full use of the limiting stress of concrete and excluding too high stresses (*Rêsal*[1]).

G. S. Williams[42] designed for the Six-Mile Creek dam at Ithaca a single form of dam in order to avoid the action of vertical cantilevers, so that all the load is carried by the arch action. The dam has at the bottom the form of an inverted dome which provides the dam, also at the bottom of the valley, with secure abutments; the water-pressure on the dome partly compensates the weight of dam.

Domes of great dimensions were used in the multiple-dome Coolidge dam on Gila River (Arizona)[43]. The dam was calculated as a system of independent arches, separated from the dome with plane sections, running perpendicularly to the inclined abutment lines.

The distribution of horizontal external forces on the system of vertical cantilevers and horizontal arches depends on the relation of the height to the total length of dam. With the increasing length of the arch dam, the length of the horizontal arches and their flexibility increases, but the vertical cantilevers remain equally rigid. Therefore the greater part of the horizontal loading acts on the vertical cantilevers and the dam gradually approaches in its statical action a straight dam of constant height, where all the load is borne by vertical cantilevers by means of compression and bending. On the other hand, in shorter dams the greater part of the horizontal load acts on the horizontal arches; with diminishing length the action of vertical cantilevers diminishes and that of horizontal arches increases. From constructed and calculated dams *Résal*[1] and *Stucky*[14] show that *the arch action is of value only in dams where the relation of the length at crownland of the height h is* $\frac{l}{h} \leqq 2.5$. Dams with $l > 2.5\,h$, where their thickness is great, are to be analysed as straight (gravity) dams. The arch action in this case can be disregarded, as it is of little importance; it is also useful for stability, because it relieves a little, especially in the upper parts, the vertical cantilevers. In relatively thin dams, even of greater length, the action of horizontal arches may be considerable[35].

Arch dams need of course secure abutments on the slopes of the valley; they can only be erected if the slopes are of solid rock. The arches at abutments should be approximately perpendicular to the contour lines of the ground.

If the analysis of the arch dam is more exact, the actual stresses calculated more in detail and the effect of temperature considered, then the limits of stresses can be (as in bridge construction) raised in respect to the ordinary brief calculation. *Stucky*[14] recommends in this case for concrete limiting stresses up to 35 kg/cm^2 in compression, 10 kg/cm^2 in tension. *Juillard*[41] objects that the working stresses used till now should not be tampered with before longer experience has proved the reliability of the new methods of analysis of arch dams.

The real stresses in arch dams can essentially depend on the mode of construction[44]. To realise the arch action it is necessary that the dam forms in vertical and horizontal direction a monolithic body; all preceding considerations as to stresses in arch dams therefore assume that during construction all layers are well connected among themselves or bound by greater stones. If there are vertical contraction joints in the dam, the arch action can be considerably reduced or even with opened joints fully eliminated. If the narrow contraction joints are afterwards filled, then horizontal transverse forces (in horizontal arches) can be transported; the friction in contraction joints produced by pressures acting on them, helps achieve this. Arch action can then be considered (at least partly).

Confirmation of the Analysis by Means of Measurements and Tests.

The stresses in a dam can be determined from measured deflection, as *A. F. Noetzli*[15] has shown. If Δs be the shortening of the centre line of arch (numerical value), then the horizontal thrust produced only by Δs is

$$H = -k_f \cdot \frac{E t^3}{h^2 l} \cdot \Delta s \tag{42}$$

for*

$$k_f = \frac{h^2 s}{t^3 \left(\int \frac{y^2 d s}{J} + \int \frac{\cos^2 \varphi d s}{t} + 3 \int \frac{\sin^2 \varphi d s}{t} \right)} \tag{42a}$$

we assume an arch with a section of the base $b = 1$, the signification of other quantities is as before (see fig. 1). The values of k_f for different central angles 2α and for different relations t/h are given by *Noetzli* in a graphical table. Approximately for a parabolic arch, if only the effect of bending moments is considered, $k_f = 0.94$; including the effect of thrusts and shears, $k_f = 0.75$. The shortening of centre line Δs gives approximately, supposing the deformed centre line to be circular (as for an arch with two hinges), the deflection at crown of arch (positive toward the centre) $\eta = \frac{3}{16} \cdot \frac{s}{h} \cdot \Delta s$. Substituting Δs from this equation in (42) we obtain

$$H = -\frac{16}{3} k_f \cdot \frac{E t^3}{h s^2} \cdot \eta \tag{43}$$

Approximately $k_f = 0.75$. $\frac{s^2}{h} = 8.3\, r$, r is the radius of centre line; that gives

$$H = -0.48 \frac{E t^3}{h^2 r} \cdot \eta \tag{43a}$$

This force H acts in the gravity axis of the centre line, therefore approximately at a distance of $\frac{h}{3}$ from the crown; it is then easy to calculate the stresses at crown and abutments of arch.

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* The numerator of k_f should be rightly $\frac{h^2 l^2}{s}$ instead of $h^2 s$.

The formulas of *A. F. Noetzli* can give, according to *W. Cain*[45], sufficiently good results for central angles 0—30° and thin arches; for greater central angles and thick arches the results will differ considerably from exact formulas.

If we consider the water pressure p (on the up-stream face) and the uniform radial loading p′ (positive toward the centre of arch) on the horizontal arch, produced by change of temperature and shrinkage of concrete, then the horizontal deflection of arch crown according to *W. Cain*[21] has a more exact value

$$y = \omega_0 r_0 \left[\varepsilon' - \delta\,\varepsilon + \frac{(p + p')\, r_2}{E\,t}\right] \tag{44}$$

with the signs used in equation (34a); for a fall of temperature, δ and p' would be negative. From (44) we can calculate $(p + p')$, the whole radial loading of arch (uniformly distributed as we suppose) if we measure the actual deflection y and other quantities in the formula. Formulas (44), (34), (34a) are valid only if the whole considered arch is in action, therefore if there are no cracks from shrinkage of concrete or from too great tensile stresses.

If there are vertical cracks in the dam which alter the action of horizontal arches (they alter the arch sections), the measurements of deflections and temperature should be made at a time when the cracks are closed; from two observations at different times one can determine as a difference in the quantities observed the deflection y and the temperature change δ, which are to be included in the formula. It is also to be considered that the unfavourable influences of unequal modulus of elasticity (of heterogeneity of concrete) and of a non-uniform distribution of temperature are excluded. To attain this, deflections and temperature should be measured over a period of several days in which the temperature of the air does not change. If the calculation from observed values gives a pressure p almost equal to water pressure at the bottom of dam or greater, it is a sign that there were vertical cracks in the dam or that the temperature in the dam was distributed too unequally; the results of such measurements cannot be used.

From the measured radial deflection η at the crown of horizontal arch one can calculate the bending moment at crown after the formula

$$M_0 = -\,a \cdot \frac{E\,t^3}{h\,r} \cdot \eta, \tag{45}$$

applicable equally for the deflection from radial water pressure as for the deflection from temperature. The coefficient a is: for a hingeles arch[21] when the influence of shear is considered,

$$a = \frac{1}{6} \cdot \frac{\sin\alpha\left(1 - \dfrac{\sin\alpha}{\alpha}\right)}{(\alpha - \sin\alpha)\left(1 + \dfrac{i^2}{r_0^2}\right) + 2.88\,\dfrac{r}{r_0}\cdot\dfrac{i^2}{r_0^2}\,(\alpha + \sin 2\alpha)} \tag{45a}$$

and for arch with two hinges[16], when the shear is neglected

$$= a\,\frac{1}{6} \cdot \frac{\sin\alpha\,(1 - \cos\alpha)}{\sin\alpha + \alpha\,(1 - 2\cos\alpha) + \dfrac{i^2}{r^2}\,(\alpha - \sin\alpha)} \tag{45b}$$

The coefficient a depends only on the central angle 2α and on the proportion $\frac{i}{r}$; a can be then calculated in advance for different angles α and proportions $\frac{i}{r}$. A numerical table thus computed gives directly a and from (45) M_0 can be determined. From this easily follows the force $X = pr - H_c$, because M_0 is the moment of this force X acting at the centre of gravity of the neutral line for an arch with fixed ends (for an arch with two hinges, the line of action of X joins the two hinges); we further compute H_c and from this force and from M_0 the stresses in the crown section of arch. From M_0 and H_c, using the formulas previously given, one can determine the bending moment and thrust in the abutment section and thus also the stresses there.

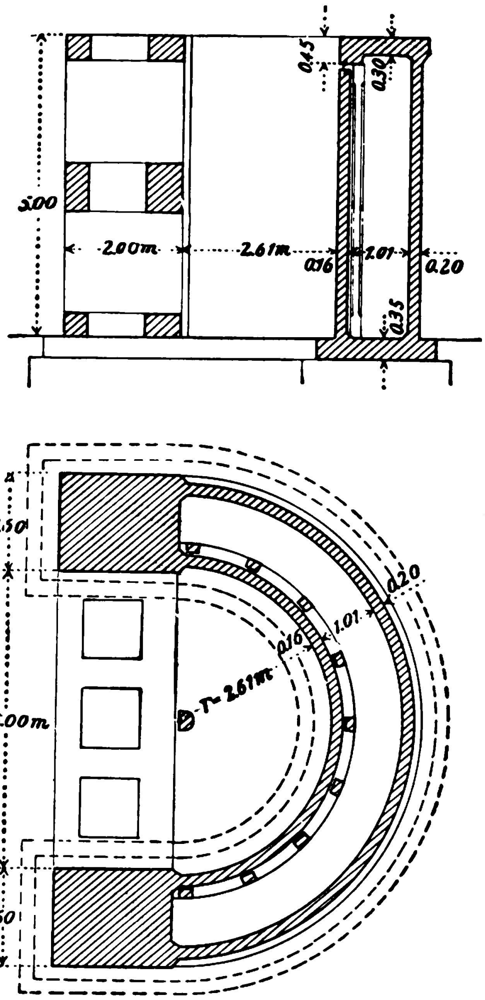

Fig. 6.

If the full action of arch is secured for all cases of loading and temperature changes, that is, if the arch dam is so reinforced and fixed to the rock at the sides of the valley that vertical cracks cannot arise the modulus of elasticity E, can be computed from exact measurements of deflection and temperature. For this it is necessary to determine by analysis of dam (according to *Smith* or *Noetzli*) the radial loading p′ of the horizontal arch; E is then given by equation

(34). It is better to eliminate the temperature change by using the observation of deflections at different depths of water and equal temperature.

The analysis can be controlled by *measurements* on actual dams which were made in a few cases[46]; one can also confirm the theory by systematical *tests on models*. *Cam. Guidi*[47] tested a model of arch dam in the form of a semi-circular arch with a radius of 2.61 m, constant thickness of 16 cm and height of 5 m; he acted on it with the pressure of water contained between this arch and a greater co-axial arch at a distance of 1.01 m, of a thickness of 20 cm (fig. 16). The two arches were fixed to the bottom and to two great lateral vertical piers; the outer arch was also fixed to a concrete plate on the top, but the inner arch ended under this plate, and between the arch and the plate there was an elastic connection to make possible the examination of the arch under pressure produced by hydraulic jacks. The experiments showed that the elastic line of the horizontal arch did not correspond to a uniform loading with water pressure, but that it corresponds to a loading with $^3/_4$ of water pressure at crown and a pressure growing continuously towards the abutments under full water pressure, as *Guidi* had recommended in his book on the statics of dams[10].

For expermients with models *A. Mesnager* and *J. Veyrier*[48] proposed in 1926 a very good method permitting the same stresses to be obtained on a model of reduced size as in the actual structure. Using instead of water a liquid with specific weight n-times greater (for instance mercury with specific weight $n = 13.6$), one obtains in a model of the same material as the actual dam, reduced in the proportion of $1:n$, at each point the same external pressures and therefore the same stresses as in the corresponding point of the actual dam; the deformations will be similar. Using for the model a material with strength m-times smaller and for loading a liquid with specific weight n, the same effect is obtained with regard to breaking in a model in a scale $1:mn$. In this way *Mesnager* and *Veyrier* tested a model of a graduated arch dam* which they designed for a total height of 70 m (5 degrees of 14 m each) on the river Dordogne at Marège[48]; they used mercury for loading (specific weight 13.6) and made the model of plaster so prepared that its strength was 7,35 times smaller than that of concrete, so that a model on a scale of $1:mn = 1:13.6 \times 7{,}35 = 1:100$ was sufficient. They loaded the model up to breaking and determined the factor of safety (from 3 to 5) of each arch in an actual structure of their design. They also found that the formulas and tables of *Guidi* are good and safe in practice. Both authors proposed to make further experiments on a concrete model on a scale of 1 : 13.6 for loading with mercury.

Tests of arch dams on a large scale were made in U. S. A., where on the suggestion of Mr. *F. A. Noetzli* an experimental arch dam was built in 1926 on Stevenson Creek in California; the experiments were described in detail in the Report (vol. I) published in the Proceedings of A.S.C.E. in May 1928. These american experiments are of the greatest importance for the confirmation of different theories because of their great extent and careful analysis of results. They gave many interesting results and showed clearly which of the theories hitherto used have any competence.

* The first straight graduated dam was designed by *Boulé* in 1894 for the dam on the Nile at Assuan, afterwards in 1912 by *Ruthenberg* in Italy.

Safety of Dams.

Straight dams generally have a small factor of safety, as a rule hardly greater than 1.5. This is proved by the accidents which have occurred to straight dams. As regards the dam of recta, the raising of the supposed water surface by 80 cm sufficed to cause failure[49], so that the factor of safety was here only a little greater than 1. Many other accidents with straight dams were also caused by a raising of the water surface above the highest level considered in planning (because of insufficiency of spillways) and by overflow of dam. The safety of dam increases considerably, without modification of its section, by curving the dam in plan; this can moreover essentially remedy the unfavourable effects of temperature changes and shrinkage of concrete, which can be met in a straight dam by using contraction joints.

The factor of safety of arch dams is, however, considerably greater than that of straight dams. This is also proved by the circumstance that accidents with arch dams are very rare and mostly caused by insufficient foundations (Moyie River Dam[50], Lake Lanier Dam[50], Gleno Dam in Italy[51]). Well constructed arch dams have a considerable degree of safety, much greater than straight dams. A straight dam resists water pressure only by the weight of masonry. If the water pressure increases only a little (by an unforeseen rise of the water level), the largest compressive stress at the down-stream face of the bottom section can be very considerably increased; the ultimate loading which would cause failure of dam, often has to the loading assumed in the design of dam, a proportion (factor of safety) little greater than 1. The curvature of dam increases its safety very considerably; a curved dam is by its form alone secure against tilting. The tests of Mr. *Mesnager* showed that the loading which the dam will safely carry can be raised several times before failure; the factor of safety given by the proportion of ultimate loading to the actual loading is here like that for other engineering structures.

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[44] *D. C. Henny* in Transactions of the Amer. Society of Civil Engineers 1915, vol. 78, p. 685.
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[47] *Cam. Guidi:* «Etudes expérimentales sur les constructions en béton armé» (Translated by *A. Paris,* Lausanne 1927, Bulletin technique de la Suisse romande).
[48] *A. Mesnager* et *J. Veyrier:* «Les barrages-réservoirs à voûtes à charge fractionnée» (Annales des Travaux Publics de Belgique 1928, p. 219—257).
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[51] Der Bauingenieur 1924, p. 382.

Summary.

Up to the present time, arch dams have been passing through a long period of development as regards their construction. Also their analysis, the first paper regarding which dates from 1866, developed from plain beginnings to the relative perfection of today. Arch dams are being more and more used and their dimensions are continually increasing; they are of eminent importance for the safety of the whole district situated below them. It is therefore interesting to follow the development of their analysis which clearly shows a more and more complete penetration of human thought into the true meaning and function of this important engineering structure.

At first the arch dam was considered as a system of independent horizontal arches, withstanding water pressure and perhaps also temperature changes and shrinking of concrete. The basis of analysis is here of course the same as for ordinary arches loaded with vertical forces. Nevertheless the nature of loading (radial) requires a somewhat different and especially a more complete analysis than the usual analysis of vertical arches. The analysis of the horizontal arch was gradually improved by considering shear, besides bending moments and thrust; further the theory of thin arches gave way to the theory of thick arches, at first approximate, afterwards exact and based on the general theory of elasticity.

Greater heights of dams led to the necessity of considering the connection of horizontal arches in a vertical direction. This is effected by analysing the arch dam as a system of horizontal arches and vertical cantilevers. The distribution of load in both systems was calculated at first approximately, neglecting the variation of loading in the direction of arch, and supposing the loading of arch to be uniformly distributed. The analysis was gradually perfected: to-day we are able to calculate the exact distribution of forces on arches and on vertical cantilevers, both for symmetrical and for unsymmetrical dams.

The last word in the theory of arch dams is their analysis as elastic shells; this idea was first worked out in practice in the United States of America and also brought to perfection there.

The American engineers working on the theory of arch dams also claim the great merit of having lately carried out large-scale experiments with a great experimental arch dam, and of having compared them with tests on small models. These experiments have cleared up many questions regarding the theory of arch dams and promise to show a safe way for their correct analysis and construction.

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VI 2

The Use of Concrete in the Construction of Solid Dams.

Über die Verwendung von Beton beim Bau massiver Staumauern.

Remarques sur l'emploi du béton dans la construction des barrages massifs.

Coyne,
Ingénieur en Chef, Paris.

I.
The Concrete Age.

"Concrete," observed Rabut in 1910, is "a masonry work of small nature composed of materials small enough to reduce the work of shaping, conveying and using to its simplest expression."[1] Compared to previous methods of building, concrete, and reinforced concrete in particular, requires "solutions of disconcerting simplicity in the varied problems of our craft, and assumes countless and often unexpected forms to meet new requirements." From his comparison, Rabut inferred that his generation was experiencing "the most profound revolution ever effected in the art of building."

An invention of the Romans, concrete has become "the modern and most economical form of masonry."[1] Requiring no rare materials or highly trained or skilled labour, adapting itself with marvellous flexibility to the requirements of design and actual construction, and being more durable than timber or steel, concrete has been adopted for all types of building construction — both ground structures and hydraulic and maritime works. "The democratic material," as Rabut familiarly termed it then, probably to signify that its development was bound up with social evolution and the gradual disappearance of the stonecutter (the wealth of another age), has become — the monarch. Its supremacy is, however, still contested by steel, especially for large bridges; but for under-water structures it has gained a veritable monopoly, and we designers or contractors would be considerably inconvenienced now if we had to do without it. In this connection, our age is truly and incontestably the concrete age.

At the present time, there is hardly a dam that is built in any other way, with a few rare exceptions, the most notable of which are the Italian dams built in ordinary masonry, due to the high quality of their workmanship (the celebrated masons of the Alp district). It is by using concrete that designers and contractors have been able to attain the colossal dimensions of certain dams erected in recent years — the largest masonry structures ever put up by the hand of man. Boulder Dam, the huge

[1] *Rabut:* Introduction au cours de construction en béton armé. 1910—1911.

Colorado barrage, actually contains 2½ million m^3, or slightly more than the volume of the Great Pyramid.

Although the conditions governing the use of concrete are constantly evolvine, even at the present time, it seems advisable to make a rapid survey and outling everything that experience has taught us about it.

II.
Ultimate Strength of Concrete.

The Cements. —

The ultimate compressive strengths required in France for dam concretes are, generally speaking, not very high; and even for gravity dams they may drop to comparatively low figures — less than 100 kg/cm² at 90 days (8 to 10 times the calculated fatigue figures, which, as we know, are only remotely related to the actual fatigue figures). For arches, the figures vary between 200 and 250 kg/cm² at 90 days, or four times the calculated fatigues.

It should be noted that the strength of concrete at 90 days (and, *a fortiori*, the strength at 28 days) is very rarely put under stress in a dam, because, as it takes several years to build, the concrete has begun to harden a long time before it is under water. Again, the newest concretes are the ones usually located at the top of the structure, and these undergo the least amount of fatigue. Consequently, the quick-hardening cements, which have been developed on a very large scale to meet the requirements of reinforced concrete construction, have no place in dams. Their strength only begins to matter after six months or even a year. On the other hand, they set up a considerable amount of heat in the first few weeks after they have set, and thus set up local or extraneous fatigue, especially when expanding, and practical men would like to obviate this.

In certain countries, particularly Sweden and the United States, there is a tendency to manufacture special cements for dams, known as "low heat cements". Generally speaking, these cements differ from the others by containing less tricalcic silicate and aluminate, which are the factors giving rapid hardening and, consequently, excessive evolution of heat. They also have the advantage of shrinking less than rapid-hardening cements and evolving less free lime, and this makes them less soluble in water. Although their initial strength is lower than that of ordinary cements, it is equal too and even higher than the latter after a twelvemonth.

Slag or metallurgical cements are commonly used in France for solid gravity dams. The low strengths necessary in this latter case call for the use of special binders which are very superior to Portland cement from the viewpoint of unreasonable heating and resistance to the effects of clean water. The only trouble with them is that their strength is comparatively low, and their hardening considerably retarded by cold weather.

Influence of $\frac{C}{E}$.

Let us consider a pure cement paste containing a weight C of cement for a weight E of water. Its strength is an increasing function of the ratio $\frac{C}{E}$ (which actually, and very nearly, measures the cement ratio of the voids in the pure paste), and experience has shown that it is connected with this ratio by a linear equation, viz. $R = \alpha\,(\frac{C}{E} + \beta)$.

Now add sand, being careful not to modify the ratio $\frac{C}{E}$ defining the strength of the binder paste, and to fill up all the voids between the sand grains with paste, without allowing air to penetrate.

At first, we suspect that the strength will not be altered, since rupture must take place due to the paste being (generally speaking) much weaker than the sand. Experiment actually shows this to be the case, as the following results prove.

The following were made up:

(1) A very fluid paste, capable of mixing easily with sand.

(2) Three mortars, obtained by adding 3, 6 and 9 volumes of sand respectively to 5 volumes of this paste. The plasticity of these mortars obiously decreased with increase in the amount of sand incorporated, but every attempt was made to get them as compact as possible, by ramming and vibrating. From each mixture, six cubes of 2″ side and six of 2¾″ side were made, and subjected to crushing tests, half of them after 7 days and the other half after 28 days.

Table 1 gives the characteristics of these different mortars at the time they were made, and Table 2 the strength figures obtained after 7 and 28 days respectively. (The mortars have been conventionally defined by the ratio of the volume of *sand* to the volume of *pure paste.*)

Now let us add stone to our mortar, still being careful not to alter the quantity of water incorporated in the cement slurry, and to eliminate, by careful moulding, all the additional voids set up by adding the stone. The strength of this particular concrete may be expected to be the same as that of the initial cement slurry, and this supposition has been verified by making the following tests.

Six cubes of 22 cm side were made up as follows (3 being tested after 28 days and 3 after 90 days):

(1) A mortar M of a very liquid type (or concrete of small mesh aggregate), containing 19% of water.

(2) Eight concretes obtained by adding to the first mortar made 30, 60, 90, 120, 135, 150, 165 and 180 per cent. of stone relatively to the total weight of dry material in the mortar, this being placed in moulds simply with the trowel or by slight ramming. (The fluidity of these concretes obviously diminished with increase in the proportion of water added.)

(3) Six concretes absolutely identical to six of the concretes mentioned above, but rammed or vibrated whilst being placed in the moulds. These concretes contained 60, 120, 135, 150, 165 and 180 per cent. of stone respectively in proportion to the other dry materials. The smaller the degree of liquidity, the more thoroughly they were rammed.

The strengths after 28 days are set out in Table 3.

Whether we are dealing with the cement slurry, or with mortar or concrete, the strength of the several samples (within the limits of error of the tests) is purely a function of $\frac{C}{E}$ and may thus be expressed by the formula: $R = \alpha\left(\frac{C}{E} + \beta\right)$,

Table 1.

Description	Quantities incorporated per Cubic Metre: Cement (C) Kilos	Sand (S) Litres	Total Weight of Dry Material	Water (Litres)	$t = \frac{\text{Vol. Paste}}{\text{Gaps Sand}}$	Compactness γ	$\frac{C}{1-\gamma}$
Cement slurry	1365.2	0	1365.2	573.4	—	0.427	0.745
Mortar 3 5	990.5	435.7	1724.7	415.9	4.5	0.584	0.745
Mortar 6/5	775,8	682.6	1926.0	325.7	2.25	0.673	0.741
Mortar 9/5	636.7	840.3	2052.7	267.4	1.5	0.728	0.732

The ratio $\frac{C}{E}$ remained constant at 2.38 (by weight) or 0.744 (by volume).

Table 2.

	Measured Strengths: Cubes of 5 cm Length of sides, 7 Days		28 Days		Cubes of 7 cm Length of sides, 7 Days		28 Days	
Cement slurry . . .	233.3		379.2		213.2		282,0	
	243.7	264.9	362.8	379.1	309.2	253.3	365.7	341.5
	317.7		395.3		237.5		376.7	
Mortar 3/5	274.5		416.0		301.5		383.3	
	260.1	256.0	399.6	390.1	231.1	277.3	336.9	366.7
	233.3		354.6		299.3		380.0	
Mortar 6/5	221.0		297.8		202.8		321.2	
	245.7	247.8	359.7	344.9	275.7	234.1	294.8	306.4
	276.6		377.3		223.7		303.3	
Mortar 9/5	247.8		366.9		218.5		309,2	
	233.3	238.1	338.2	343.7	292.7	259,1	347.9	340.0
	233.3		325.9		266.1		380.8	

where α und β only depend on the cement, the method and time of hardening, the mould, and the dimensions of the test samples.

The strength of a *compact* concrete, i. e., one containing no other interstices than the ones in the pure paste, is therefore nothing more or less than the strength of this pure paste.

	Concretes made	Quantities incorporated per m³ of Concrete								Density of fresh Concrete	Fluidity measured at the jig	t = Vol. of Mortar / Voids in Stone	Compactness γ Factor Concrete		C actual / 1 − γ actual	γ actual / γ theoretical	C / E (By volume)	Strength at 28 Days	
		Cement C (kilos)	Sand S		Stone P		Total Weight of Dry Materials	Water E											
			Weight (Kilos)	Volume (litres)	Weight (Kilos)	Volume (litres)		% Weight of Dry Materials	Volume (litres)				Actual.	Theoretical				On 3 cubes 22 × 22 cm for Mortar or Concrete	Mean
1	Mortar M *normal*	461,0	1862,5	856,2	—	—	1828,5	19,0	346,5	2,170	non-determined greater than 2,20	—	0,651	0,654	0,416	1	0,416	225,9 — 228,8 — 188,5	207K,7
2	M + 30% *Normal* aggregates	381,9	1128,6	709,8	453,2	306,6	1963,7	14,62	287,0	2,251	non-determined greater than 2 20	6,14	0,713	0,718	0,416	1	0,416	187,0 — 202,5 — 208,8	199,4
3	M + 60% *Normal* aggregates	325,4	961,8	604,9	772,3	522,5	2059,5	11,88	244,6	2,305	do.	3,07	0,753	0,755	0,414	0,997	do.	219,7 — 221,2 — 183,8	208,2
4	do. *vibrated*	325,9	968,3	605,9	773,5	528,2	2062,7	11,88	245,0	2,308	do.	(1)	0,755	—	0,416	—	do.	171,4 — 183,8 — 174,5	176,6
5	M + 90% *Normal* aggregates	284,0	839,2	527,7	1011,0	684,0	2135,2	10,0	213,5	2,349	2,20	2,05	0,784	0,786	0,413	0,997	do.	180,7 — 211,9 — 191,8	195,8
6	M + 120% *Normal* aggregates	250,8	741,3	466,2	1190,6	805,5	2182,7	8,63	188,5	2,371	2,05	1,53	0,807	0,811	0,406	0,995	0,416	202,5 — 188,5 — 177,6	189,5
7	do. *rammed*	252,1	745,1	468,6	1196,5	809,5	2193,7	8,63	189,3	2,383	2,05	(1)	0,811	—	0,416	—	do.	183,8 — 187,0 — 199,4	190,1
8	M + 135% *Normal* aggregates	236,6	699,3	439,6	1263,4	854,8	2199,3	8,09	177,9	2,377	1,92	1,86	0,814	0,821	0,397	0,991	do.	187,0 — 194,8 — 190,1	190,6
9	do. *rammed*	238,6	705,4	43,6	1274,2	862,1	2218,2	8,09	179,5	2,398	1,92	(1)	0,821	—	0,416	—	do.	171,4 — 180,7 — 194,8	182,3
10	M + 150% *Normal* aggregates	228,8	661,4	416,0	1327,8	898,4	2213,0	7,60	168,2	2,381	1,80	1,23	0,820	0,830	0,388	0,988	do.	205,7 — 187,0 - 194,8	195,8
11	do. *rammed*	225,3	665,8	418,7	1336,7	904,3	2227,8	7,60	169,3	2,397	1,80	(1)	0,826	—	0,405	—	do.	215,0 — 187,0 — 194,8	198,9
12	M + 165% *Normal* aggregates	212,1	627,0	394,8	1384,5	936,7	2223,6	7,17	159,4	2,883	1,72	1,11	0,825	0,839	0,379	0,983	do.	201,0 — 191,6 — 185,4	192,7
13	do. *rammed*	213,6	631,4	397,1	1394,3	943,4	2239,3	7,17	160,6	2,400	1,72	(1)	0,831	—	0,395	—	do	216,6 — 199,4 — 193,2	203,1
14	M + 180% *Normal* aggregates	201,5	595,5	374,5	1434,6	970,6	2231,6	6,78	151,3	2,388	1,65	1,02	0,829	0,846	0,368	0,980	do.	202,5 — 191,6 — —	197,1
15	do. *rammed*	203,9	602,6	379,0	1451,8	982,3	2258,3	6,78	153,2	2,412	1,65	(1)	0,839	—	0,395	—	do.	202,5 — 205,7 — 218,1	208,8

(1) *This ratio con Electuale after ramming in the forms; it could not be verified.*

It will be noted that, on the tests, the percentage of cement varied from

100 p. c. (pure paste) to 0.9 p. c.

by weight of the dry materials, and the compactness from 0.427 to 0.839, without affecting the strength.

On the other hand, the strength takes an upwardt tendency from a certain percentage of stone onwards, due to the fact that the pure paste is then reduced to a thin film occupying the very limited space available between the stones imbricated in each other.

At the limit, a masonry of dry stones would be reconstituted, and this would be endowed with strength although it did not contain the slightest trace of binder.

Thus the conclusion to be drawn from the tests would be that the hardness of the concrete varies in inverse proportion to the amount of cement it contains, which is an elegant and unexpected way of reconciling safety with economy.

Practical Considerations. —

These paradoxical conclusions call for explanation. Obviously the most compact of our concretes, and especially the last ones in the list, represent an ideal grouping of the materials, the largest stones making contact with each other and their voids being filled up to just the right extent with mortar.

No building works ever produced such a concrete. It is a work of art, rather like a broken plate or a kind of jig-saw puzzle, the pieces being put together by an expert hand so as to restore the plate and reduce to the minimum the number of gaps in the structure. There is no reason why the pieces put into a concrete mixer and shaken up for any length of time should automatically fall into the place assigned to them by the hand of the artisan. Suppose this were possible, there is every chance of the concrete de-mixing on its journey from the concrete mixer to the place where it has to be used, and the strength would no longer be a function of $\frac{C}{E}$, but of $\frac{C}{E+V}$, V being the voids.

It is very strange to note that the majority of research workers who have claimed to give rules for producing the most compact type of concrete, have overlooked the fact that their concretes were no more capable than our reconstructed plate was, of being transported from the place of mixing to the site without risk of demixing, and that the conclusions derived from their laboratory tests were dependent on very ticklish rules which vary with the skill of the operator.

The foregoing example should be sufficient to prove this. The maximum compactness of a concrete is usually supposed to be obtained, even in the laboratory, by increasing by 30 to 35 p. c. the volume of mortar strictly necessary for filling up the voids between the stones.

Of our concretes, the most compact is the one in which the pieces touch, and the ratio t between the mortar and the voids in the stones is scarcely more than 1, as was expected theoretically.

One necessary conclusion, then, is this: all the laboratory tests made on the compactness of concretes should be accepted with a great deal of reserve, since they have only a very remote connection with the actual conditions under which the materials are arranged at the building sites.

In this particular respect, the concretes differ considerably from the mortars.

Working Properties.

The best concrete for a particular purpose can therefore only be defined at the particular time it is being used, or else by an experiment carried out on a scale large enough to repeat the exact conditions of practice. To improve our knowledge, Bolomey is the first who has given to the practical men really practical advice, based on experience in actual works. His advice will, however, be found to be precisely opposite to the conclusions which we might be tempted to deduce from laboratory tests, so that the latter just be interpreted the other way round to ensure good work.

Before everything, a concrete must be *well prepared* to be compact on the job, especially on a dam where labour or workmanship is necessarily limited owing to the quick rate of working.

Compared to the best laboratory concrete (or to our ideal conception regarded as a basis of comparison), the concrete must include a surplus of water, a surplus of sand, and a surplus of fine sand. But all these things conflict, or seem to conflict, with strength.

The function of the surplus water is to convey and lubricate the materials so as to facilitate their grouping, the excess of sand acts in the same direction and also serves to make up for the local deficiencies which would certainly arise if the quantity of sand were just sufficient to fill up the voids between the stones. But these two precautions would not be sufficient to obtain a compact concrete on the job, for the water has a tendency, whilst moving, to separate from the concrete and to carry part of the cement along with it. It must be fixed by introducing into the mixture a sufficient quantity of fine dust, viz., fine cement or sand, which fixes the water by capillary action and prevents segregation. In this way, a smooth, binding paste is formed — a kind of emulsion in which the stones float, and which only breaks slowly when once the concrete is in place, expelling part of its surplus water by the effect of the pressure exerted by the successive layers.

It is these cautious remarks which have led Bolomey to suggest the "granulometric curve" which bears his name, and regarding which we shall merely make a few observations.

As these are purely practical problems for which it is impossible te give any general solution, Bolomey's curve is only suitable for certain classes of materials and certain methods of working. Generally speaking it needs, for broken materials, to be corrected by increasing the sand and the fine sand, as shown below (Plate 1).

It may, however, have to be departed from considerably for special forms of materials, such as the broken porphyrys, diorites and quartzites. These very "cold" stones break up flabily in all stages of granulation, and include a large number of voids which have to be filled up with large surpluses of sand.

The added Sand. —

In this latter case, the use of added sand — dune or river sand in round grains — is indispensable, because its shape makes it act as a lubricant to allow the broken material to arrange itself properly, and prevents the pieces merely supporting each other.

We have been able to utilise granulometric compositions comparable to the following (Plate 2.), with rich concretes intended for use in a thin-vault dam (300 kg of cement per m^3 of work; maximum diameter of the stone = 70 mm).

The use of large stones (above 10—12 cm) makes the concrete less workable, and it may also probably be necessary in this case to depart from Bolomey's empiric standards. At any rate, wide tolerances are necessary, to make up for the unavoidable variations in the supply of materials, since perfection in this particular respect is impossible.

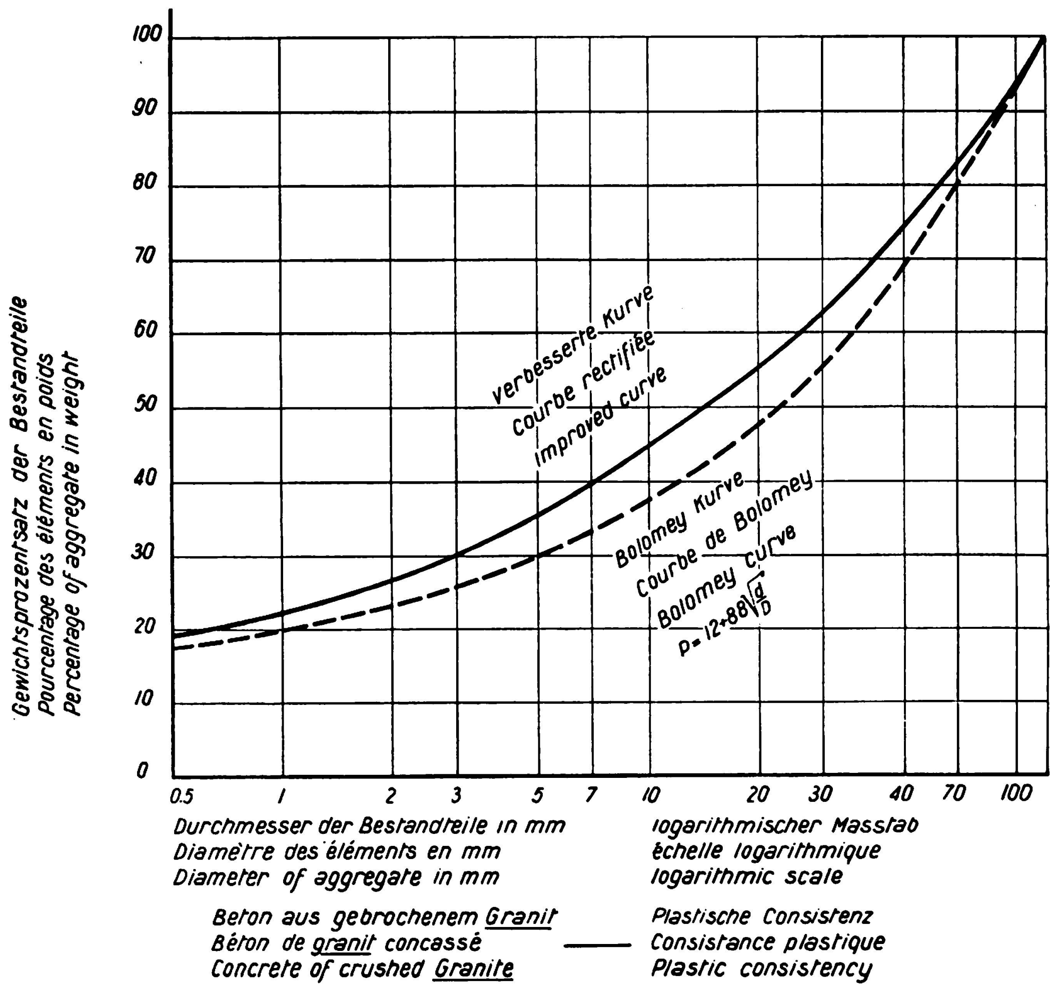

Fig. 1.

It will be seen, then, how difficult are the problems met with on the actual job, and how important it is that these difficulties be properly tackled by the works manager according to the circumstances which arise.

The main thing is that fluidity and grain-composition (grading) should be such as to give the concrete a natural tendency to go properly and automatically into place; and to acquire of itself the requisite homogeneity and compactness, seeing that workmanship is necessarily limited to the rate of working, and reduced to a few detail corrections.

For this reason, the concrete should exhibit no tendency to de-mix (a factor dependent on the way it is put in), and its external friction should be a minimum. This twofold condition in decisive for the handling — previously the attribute of plastic concretes only. Certain products improve it, particularly kieselguhr, or metallic salts incorporated in the mixing water.

It should be noted that, assuming equal fluidity, a "workable" or good handling concrete will be easier to mix than another. An easy way of checking this is to measure the torque at the shaft of the concrete mixer, as is done in America.

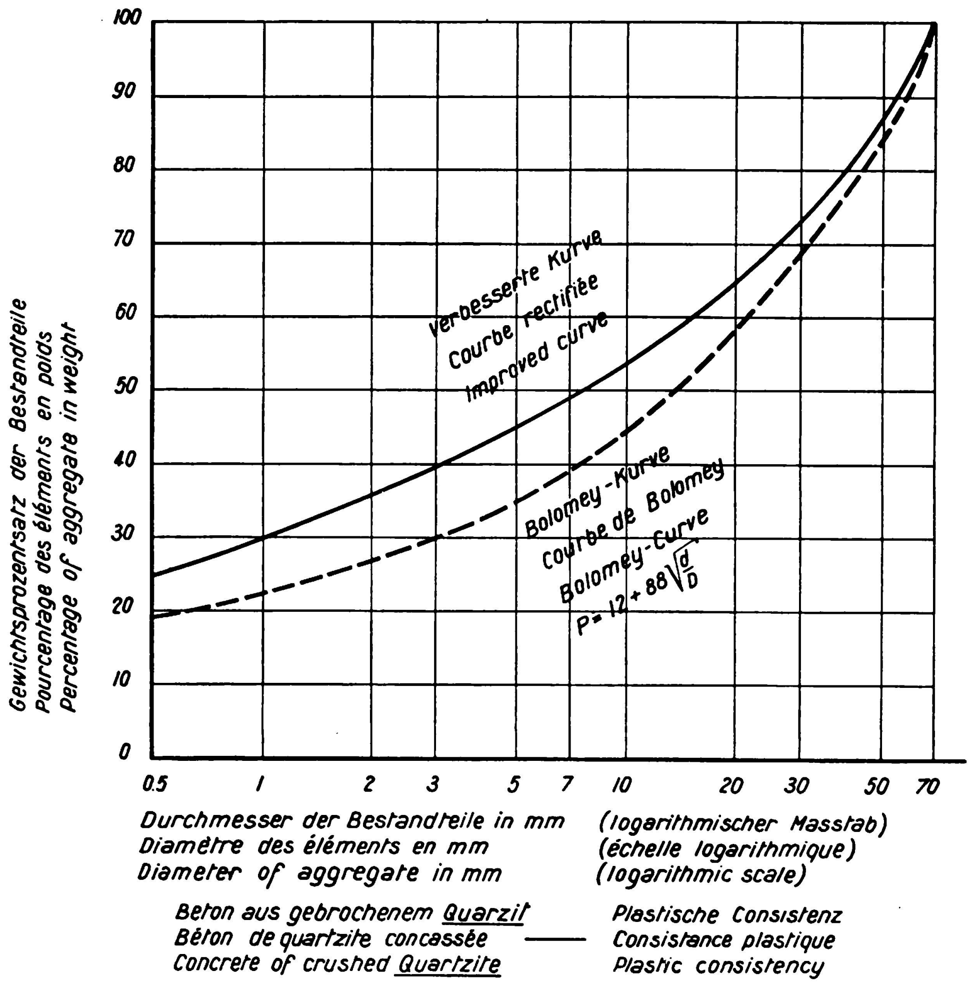

Fig. 2.

Vibrating.

As vibrating is an artificial method of compacting, but one which rightly shows an increasing tendency to be adopted at all building works, it is certain that, for the reasons outlined above, it must call for a special concrete different from the concrete made by ordinary methods. For small parts made of plain or reinforced concrete, it will be noted that the optimum granulometry (grading) of vibrated concrete differs widely from that of ordinary concrete. It is not impossible to obtain in this way concretes which are very dense and contain a high percentage of stones, and which resemble our broken plate or the jig-saw puzzle which we used by way of comparison. The dry concretes, particularly those graded discontinuously and including in their composition a minimum of sand, give very high strengths when vibrated.

On the other hand, ordinary concretes, and particularly cast concrete, desintegrate under the prolonged action of the vibrator which, while assisting the compacting and settling of the concrete, cause the excess of mortar and of pastes to flow up to the surface, thus dividing the mass into alternate layers of very hard concrete and diluted mortar.

Referring to the table on page, it will be noted that concrete No. 4, which is fluid and was vibrated in the test mould, is not so strong as the same concrete which was not vibrated.

Unfortunately, things are very different at a barrage works.

What actually ought to be done is to really vibrate the concrete and to vibrate it everywhere.

Here, again, the rate of working makes the job troublesome, because the vibrators absorb a great deal of power, their scope is comparatively small, and there is a limit to the number of apparatus that can be set to work. The power required is greater, and the radius of action smaller, the less "workable" the concrete.

If too much be expected of vibrators, and it be decided to manufacture a dry and "stiff" (i. e., not very workable) concrete, there is a big danger of disappointment, especially as regards impermeability. Experience almost everywhere has proved this.

In the present state of things, the adoption of the vibration method at a dam construction works can only be regarded as a simple adjunct for compacting the concrete, especially in the outer zones, for increasing impermeability. The grading of the cement will be practically the same as for non-vibrated concrete. At the same time, the quantity of water may be slightly reduced.

Under these conditions, no attempt must be made to vibrate the concrete thoroughly, for fear of disorganizing the mass and causing the liquid constituents to flow back to the surface of each vibrated layer.

The best vibrators are the internal type, and their sphere of action is directly proportional to their frequency, assuming equal power. The most efficient vibrators at present are those revolving at 8,000 r. p. m.

Progress may possibly be made shortly in the vibration method as applied to the construction of dams, but the writer thinks it will always be advisable not to trust exclusively to vibrators and to allow for a certain automatism in operations, f only to make up for the deficiencies of the men.

III.
Technical Control.

Preliminary Tests. —

The foregoing observations show that the preliminary tests are only valuable as a first approximation, and that they must not be relied upon too much for defining in advance the granulometry curve which is applicable on the site of operations.

Control just before Use.

The workability or "handling properties" of concrete, which is an essential element of its compactness, can only be assessed when it is about to be used, and, generally speaking, by eye. No machine can replace the experienced eye in this connection.

The apparatus used for measuring the quantities of the different classes of materials (usually 2 parts of sand, and 2 or 3 of gravel) incorporated in each mixing must be accurate and reliable. Percussion type volumetric testers usuully employed for estimating the material require careful supervision. The automatic weighing machines, which have not been generally adopted in Europe for coarse materials but which are already employed in America, are much more preferable.

Once the plant has been properly adjusted, the chief place where control must be exercised is at the concrete mixers, where it is easy.

The quickest and most practical method of checking the quality of the concrete actually obtained is to measure the *density* of the fresh concrete, and this can easily be done by means of a box of known volume and a simple Roman balance (steelyard). Generally speaking, variations in the density of the fresh concrete will be found to be fairly closely related to variations in the compressive strength. This simple method supplements the visual test of the concrete and enables all defects in composition to be disclosed at once.

Tests on Mortars.

As mentioned by Bolomey, it is possible to check the strength of the concrete by tests on cubes of mortar drawn from the mixing.

The strength will usually be found to be 10 to 20 p. c. higher on the mortars (7 cm cubes) than it is on the concretes (30 cm cubes). This factor appears to conflict with the conclusions of the previous section of this paper, but it is merely due to the fact that part of the water of the concrete is retained by the large stones when the mortar is withdrawn from the mixture.

Under these conditions, the $\frac{C}{E}$ of the mortar is higher than that of the concrete, which explains the increase in strength observed.

Tests on Concrete. —

These tests on mortars, which are very practical (especially for checking uniformity in the quality of the cement), and which can be carried out at no great cost, do not dispense with the need for making compactness and strength tests on the actual concrete. The smallest linear dimension of the cylinders or cubes of concrete used must be at least $2\frac{1}{2}$ times that of the largest stones. The unit strengths appear to increase slightly with the dimensions of the test pieces (for equal $\frac{C}{E}$, of course), probably due to the better compactness of the materials by the effect of their weight.

Concrete must be tested not only for its ultimate compressive strength, but for its tensile strength as well, as the latter is important in view of preventing cracks. The best way of making the measurement is by the bending test at a constant moment.

Crushed sands, so long in disrepute because of their bad workability, give tensile strengths 10 to 20 p. c. higher than the figures obtained with (natural) round sands. Where the latter are only slightly clayey, their tensile strength becomes very low, and this shows how important it is carefully to wash alluvial materials.

The ratio $\frac{\text{compressive strength}}{\text{tensile strength}}$ which measures brittleness, varies from 8 to 12, according to the age and hardness of the concrete. It is higher with increasing age and hardness.

Controlling Concrete on the Job. —

It is no use depending on all these laboratory tests for assessing the actual quality of the concrete on the job. To get an accurate idea in this connection, repeated samples must be taken from the actual mass of the dam even some considerable time after the concrete has set.

For cast concrete, the actual strengths of the finished material are usually found te be much higher than the strengths of test cubes. In one specific case, the test strength was round about 100 kg/cm^2, whereas on the actual job the figure was twice as high at the same age.

This phenomenon is general, and is due to the fact that the enormous surpluses of water in the cast concrete cannot get away before the concrete sets, in the test sample. On the actual job, however, decantation takes place for the first few hours and is considerable compared with the thickness of the layer, with the result that there is a greater amount of heaping due to the effect of hydrostatic pressure.

At the Oued Foddah Dam, concrete mixed in the concrete mixer with 210 litres of water per m^3, had an actual compactness factor on the job of 0.84, a figure equivalent to a maximum of 160 litres of water, which means that 40 to 50 litres, or 25 p. c. of the water fed to the mixer was expelled.

Mixed with 160 litres of water, the concrete would practically have been in a reasonably plastic state but would have taken a great deal of care to put in. The surplus water added to the mixer made it easier to transport and put the concrete in place without reducing its quality.

Influence of $\frac{C}{E}$ *with Time.* —

It should be noted, however, that the effect of surplus water decreases in terms of time, as will be seen from the table below:

Proportions	$\frac{C}{E}$	Compressive Strengths (in kg/cm^2)				Remarks
		7 Days	28 Days	90 Days	1 Year	
Cement 250 kg	1.504	165.1 *167.2*	225,1 *226.2*	248.3 *261.6*	291.6 *299.0*	Plastic Concrete
Water 166.2 Litres		166.2	225.6	254.9	295.3	
Cement 250 kg	1.160	103.1 *108.6*	146,0 *167.2*	176.9 *183.3*	227.3 *237.2*	Cast Concrete
Water 215.5 Litres		105,8	156.6	180.1	232.3	

The strengths of cast concrete increase more rapidly than those of plastic concrete. Taking the ratio $= \frac{\text{Strength of plastic concrete}}{\text{Strength of cast concrete}}$ for different periods of crushing, the following figures are obtained:

After 7 Days = 157%
,, 28 ,, = 144%
,, 90 ,, = 141%
,, 1 Year = 127%

For plastic concrete, even when vibrated, the differences between test samples and concrete as actually laid appear to be much less; and, provided it be not too soft, it is not at all certain that its strength as laid exceeds the strength of cubes taken from the mixing.

Nevertheless, the ultimate compressive strength will generally be higher than what is called for by the coefficients of safety usually adopted for solid dams, which would mean that the proportion of cement could be reduced.

As will be seen farther on, there is great danger in reducing the proportion of cement, especially at the face of the dam, because of attack by water or atmospheric agents.

The high temperatures at which the concrete of a dam is maintained after setting have a definite effect on hardening. They accelerate it in the first few weeks, but this does not imply that they do not retard it afterwards.

The Weak Points: Work joints and cracks. —

The work joints are the weak points in a dam as regards shearing strength and impermeability. Accumulations of stones form there, while leakage through the mass often takes place (in horizontal work joints).

Two precautions are usually adopted: (a) Reopening the surface of the work joints so as to bring away the layer of light constituents which are often very thick in the case of cast concrete, and to expose the accumulation of stones. (b) Spreading a layer of mortar or fine concrete a few centimetres thick over the joint when work re-starts again.

This latter precaution, the author thinks, is indispensible for preventing leakage. If absolutely necessary, and where there is no accumulation of laitance at the surface, method (a) may be replaced by spraying with a jet of water and compressed air a few hours after the concrete has been put in and while it is still soft enough to be defaced slightly.

The shrinkage of dam walls is relatively slight. In the case of one fairly large block, the writer found that it still contained a large proportion of free water even several months after it was cast, and that it was bordering on the state of saturation. This water kept the concrete soaking and so practically eliminated the shrinkage due to drying, or reduced it to a very small amount, 1/10,000 th at most. The thing to do is simply to prevent the faces drying up while the concrete is beginning to harden.

Contraction in dams is therefore almost entirely a phenomenon of heat contraction. The measures adopted to allow for this are known, and actually consist in cutting the work up into separate voussoirs or blocks. Experience shows that these blocks should not exceed 15 metres in length. Unfortunately these precautions are not sufficient to prevent longitudinal cracks developed in gravity dams as shown in the illustration below, and which are the most dangerous.

It also frequently happens that the faces of the blocks in course of construction are subject to cracking in all directions owing to the difference in temperature between the inside and the surface.

Owing to a stoppage at the works whilst the Oued Foddah Dam was being buitl, a large block of concrete 35 × 35 metres at the base and 12 metres high remained exposed to the air for 18 months. At the end of 6 months the block was found ot

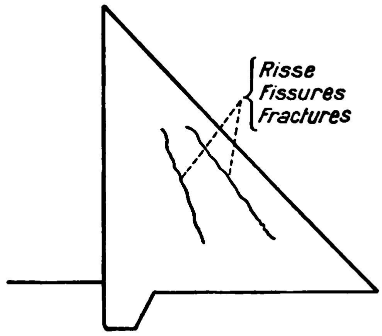

Fig. 3.

be traversed not only by two vertical cro sed fissures, but also by a horizontal fissure encircling the block mid-high. This phenomenon is easily explained. Shrinkage took place more rapidly towards the surface. Since the centre core did not contract to the same extent, it put the outside in tension and caused it to crack.

Hence the interest of the measures adopted at Boulder Dam for cooling the blocks and artificially setting up a thermal state of equilibrium which could only be reached naturally after several decades. Once this state is created, the shrinkage cracks set up in the mass should of ecurse be grouted, and the author thinks it should be possible te regrout them two or three times.

We therefore see how wrong it is to regard the mass of a large dam as a monolith, and the caution that the practical man must observe in trusting to calculations based on the homogeneousness of the mass.

IV.

Impermeability and Resistance to the Action of Water.

Impermeability. —

It is an actual fact that cast concrete is by far the most watertight material that can be used for the construction of a dam. The opposite obtains in the laboratory, where dry concrete is the most impermeable and where cast concrete is worthless. This contradiction once more reveals the fundamental importance of the conditions governing construction.

As was pointed out earlier on, for cast concrete used in large masses, 40 to 50 litres of water per m^3 are automatically expelled, this water having merely acted as a vehicle and lubricant for the concrete and guaranteed it against all local dificiencies.

The reverse is the case with plastic concrete, and, a fortiori, with dry concrete. On the actual job they frequently contain, especially at the work joints, accumulations of pebbles which form waterways.

Another cause is, however, responsible for the impermeability of concretes put in with an excess of water. This has recently been revealed by *M. Mary* of the laboratory of the *Ecole Nationale des Ponts et Chaussees*, Paris.[2] Actually, if a test sample of concrete is subjected to a filtration test at constant pressure, the volume filtered decreases very rapidly with time, even where the water is distilled. This is caused by the concrete swelling in the water — a fact which confirms the very low permeability of concretes kept under water.

Dissolution of Lime. —

The same quantity of filtered water also carries away much less limes in concretes kept under water than it does in concretes kept in air. This fact explains the importance of keeping dam concretes in a state of permanent imbibition, and to a certain extent justifies the surplus water in the cast concrete. The surplus gives a concrete that is less permeable and, for the same permeability, less soluble.

Depositis of Lime. —

Waters containing lime set up a surface deposition of carbonate of lime, which is very effectual in reducing the volume of filtered wâter which, nevertheless, comes out charged with lime. After having deposited its carbonate of lime on the face of the dam, the water is again attacked by the lime of the cement it has dissolved.

This phenomenon only goes on for a certain time.

With pure water, on the other hand, the phenomenon continues, and even laboratory tests have shown it to be aggravated with time, by gradual increase in the filtered discharge, which testifies to a slow dissolution and destruction of the structure of the concrete.

Deposits of Vegetations. —

Very fortunately, phenomena of natural deposition or colmation take place on the dam, due either to matter in suspension in the water, or to the abundance of algae on the walls and even in the fissures of the concrete.

Minimum Proportion of Cement. —

The concrete is much less vulnerable when it contains a high proportion of cement (250 to 300 kg/m^3 at least at the face of the dam, depending on the dimensions of the stones). Very serious decomposition may be expected where the mixture is a "lean" one. There is thus the risk of grave danger in reducing excessively the proportions of cement in the concrete, especially on the upstream side of the dam.

As was pointed out previously, slag and metallurgical cements are much less soluble than Portland cement.

Grain-size of Fine Sand. —

The grade of fine sand used (e $\leqq$ 0.5 mm) has a considerable bearing on impermeability, which may be considerable increased by pushing up the proportion of very fine sand, or, better still, the proportion of cement.[4] Above 0.5 mm, the grain size has

[2] See *Annales des Ponts et Chaussées*, May-June, 1933, and November-December, 1934.

no effect on the watertightness of the structures, except that it makes the concrete easy to handle.

Kieselguhr in the proportions usually employed (2 to 3% the weight of the cement) only has an indirect bearing on this particular question, by keeping the concrete compact and casy to handle.

Resistance to Erosion. —

As regards anti-erosive properties, concrete is capable of withstanding, without damage, the contact and even the impact of water moving at very high velocities, say, up to 25 m/sec, provided it is rich in cement, and that its surface is perfectly smooth and free from fissures. In America it has even been found capable of standing up to a much higher speed of flow (up to 50 m/sec.) provided the direction of the streamlets is parellel to the surface of the concrete and that the latter is well finished and very smooth. In that case it will be wise to reinforce the concrete. The discharge galleries of the Mareges Dam are arranged as follows: a steel pipe traverses the dam, forming a strong and watertight element. To prevent it corroding, it was sprayed with a coating of mortar applied with a cement gun, a light metal fitting being welded to the conduit to ensure the mortar adhering properly.

The last few inches of the coating are of carborundum mortar so as to withstand erosion better, and are reinforced by a grillage. The speed of the water is roughly 20 m/sec.

Resistance to Climatic Effects.

Climatic effects, i. e., alternating periods of heat and cold, and especially frost, have been found to cause serious trouble at the faces of dams. Then engineers noticed that similar trouble was experienced in structures erected at places where the climate was mild, and finally realised that such trouble and damage were not caused simply by the rigours of the climate, but also by defects, or bad work.

By "defects" is meant the lack of homogeneity and compactness in the concrete (accumulation of the binders at the work — joints (joints de reprise) in the cast concrete, nests of stones, and, particularly, an insufficient proportion of cement). On the pretext of economy, some engineers have even dropped to proportions of 125 and even 100 kg/m^3 for the body of the work, and barely reached 150 to 200 kg/m^3 for the faces of the dam.

Experience proves that such concretes are very open to attack by the atmosphere and particularly prone to attack by frost. The best remedy for this trouble, in a severe climate, is to include a higher percentage of cement in the concrete for the face-walls. There need be no fear of going up to 300 and even 350 kg of cement per cu. metre = of work if we want to be sure of escaping all trouble of this kind.

It is curious to note that all the damage mentioned has been on solid dams. Reinforced concrete dams (except the Gem Lake barrage, where the concrete was badly made) generally give excellent results, even the Suorva Dam, which is located in Sweden in the polar circle and whose thickness at the face-walls is only a few decimetres. This clearly proves that the trouble is due to errors in the composition of the mixture.

Conclusions.

In the present state of technical progress, the qualities required of a concrete for the successful construction of a solid dam is good handling properties which will enable the concrete to put in almost automatically. This means adopting *"plastic"* concretes.

The quantities of mixing water, sand, and fine sand are therefore fixed to meet this first requirement (depending on the materials available and the method of working), and so give the composition.

Since, however, the composition required to give the necessary strength is fairly low, a composition with a higher percentage of cement must be employed on the faces (walls) so as to guarantee the structure against the risks of leakage, decomposition by water, and attack by atmospheric reagents. This composition will be 250 to 300 kilograms of cement per m^3 of work put in, at least on the upstream wall. It is well to go to 300 or 350 kilograms on the two walls if the climate is severe.

To prevent excessive heating up, a special cement must be manufactured for solid dams, and, for very large sections, artificial cooling.

In the present state of technical development, vibration does not enable good workability to be dispensed with. At the same time, it is a valuable adjunct for working and increases impermeability in the upstream zone. It is to be hoped that its adoption will become general in the form of highpower high-frequency internal vibration.

Special precautions should be taken at the horizontal work-joints (by carefully opening up the work and applying a layer of mortar or fine concrete at the moment when work is re-started again.

The only reliable method of control, both for strength and impermeability, is by controlling the concrete on site, on samples drawn from the actual mass of the dam.

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VI 3

Reinforced Concrete Piles During Driving.

Das Verhalten von Eisenbeton-Pfählen während des Rammens.

Le comportement des pieux de béton armé lors du battage.

W. H. Glanville
D. Sc., Ph. D., M. Inst. C. E., M. I. Struct. E.
and
G. Grime, M. Sc.,
Garston.

Introduction.

In and around London there are many building sites where the ground consists of alluvial or made-up soil of very low bearing power for perhaps 10 ft. to 30 ft. from the surface. Below this a stratum of hard, compact gravel is to be found of varying thickness from perhaps a foot or two to 20 ft., a variation of this order occurring over any one site. Below the gravel a stratum of comparatively soft earth of low bearing power is again found and at a greater depth still a hard, compact clay is reached. In designing structures for such sites the engineers, owing to the uncertain thickness of the gravel belt, have in many cases thought it advisable to found below the gravel on the hard clay. In penetrating the gravel very hard driving conditions are experienced, and difficulties have been found in constructing precast piles of sufficient strength to withstand the very severe conditions. Typical examples of failures are shown in Figs. 1 and 2.

Before the commencement of the investigation very little information was available as to the effect of driving conditions upon the behaviour of the pile and no satisfactory method existed for determining the correct weight of hammer, height of drop or amount of head packing for a given pile. The rough rules provided by experience led to unsatisfactory results, and from the very limited knowledge available it was impossible to decide whether or not trouble would be likely to occur. For the engineers and for the contractors this state of affairs was very unsatisfactory. The Building Research Station was approached by the Federation of Civil Engineering Contractors and undertook to carry out an investigation into the behaviour of reinforced concrete piles with their collaboration.

The main problem of the investigation was, therefore, to devise methods of estimating the driving that a pile would stand without damage. This involved (1) an examination both analytically and experimentally of the nature and magnitude of the stresses induced in piles by impact, (2) a study of the effect upon impact resistances of the methods employed in the design and manufacture of the pile, (3) the development of methods of indicating dangerous conditions during driving. A full account of the investigation will appear as an official publication from the Building Research Station. An abridged account, rather fuller than that given in the present paper, has already appeared as a paper in the Journal of the Institution of Civil Engineers[1].

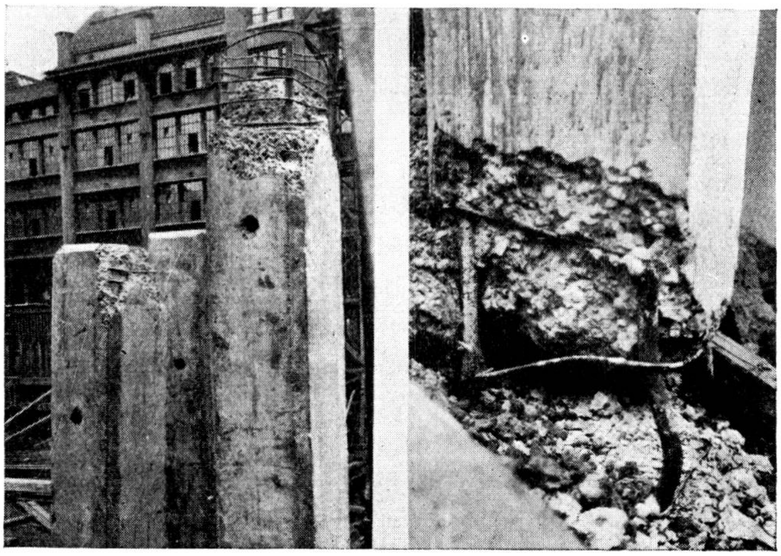

Fig. 1.
Examples of failure in reinforced concrete piles.

Theoretical Considerations.

General.

It is impossible within the scope of the present paper to attempt to give the full mathematical investigation that has accompanied the research. For a fuller account reference should be made to the Journal of the Institution of Civil Engineers[1] and to the Building Research Station publication.

Early in the study of the problem it was realised that useful representation of the conditions of pile driving could only be made on the basis of the wave theory of the propagation of stress in elastic rods. For the purpose of analysis the following assumptions are made: —

a) That the pile is undamaged when driven.
b) That the pile behaves as a linearly elastic rod.
c) That stress-waves in the hammer may be neglected.
d) That the dolly, helmet and packing are equivalent to a spring which will be referred to as the cushion, through which the compression is propagated instantaneously.
e) That the foot-resistance is elastic, the foot-pressure being proportional to the downward foot-movement. The method of relating actual cushions and foot-resistances to these ideal conditions is given later.

[1] „The Behaviour of Reinforced-Concrete Piles during Driving." By William Henry Glanville, D. Sc. Ph. D., M. Inst. C. E., Geoffrey Grime, M. Sc. and William Whitridge Davies, B. Sc. (Eng.), Assoc. M. Inst. C. E. (J. Inst. Civ. Eng., December, 1935).

The equation generally applicable to wave-motion in a long, thin, linearly-elastic rod has a general solution of the form

$$\zeta = f\left(t - \frac{x}{a}\right) + F\left(t + \frac{x}{a}\right),$$

where ζ denotes the displacement of the cross section from the initial position,
t „ „ time (after beginning of impact in this case),
x „ „ co-ordinate of any cross section measured from one end (the head of the pile),
a „ „ velocity of longitudinal waves in the rod.

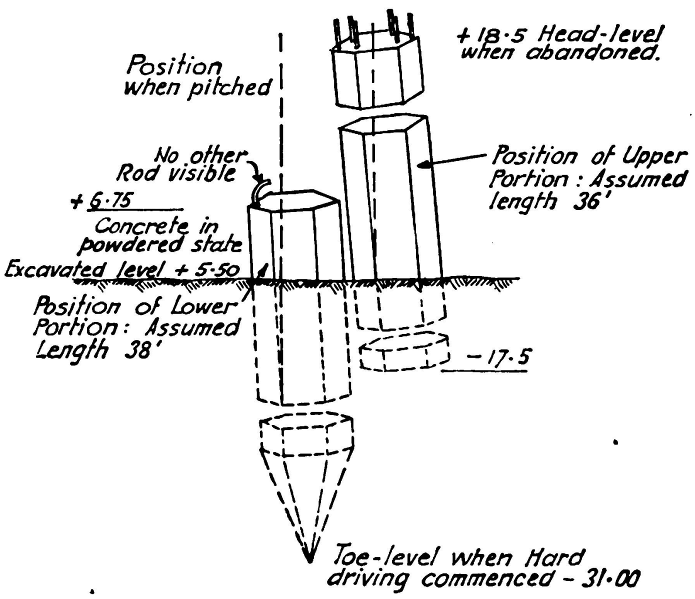

Fig. 2.

Failure at centre of pile.

The equation states that the displacement of any cross section is obtained by adding together two functions, the first of which, $f\left(t - \frac{x}{a}\right)$, represents a wave travelling down and the second, $F\left(t + \frac{x}{a}\right)$, a wave travelling up the pile.

There is no wave in the upward direction until a time $\frac{l}{a}$ (where l denotes the length of the pile) has elapsed, that is, until reflection has taken place from the foot, and this wave will not reach the head until a time $t = \frac{2l}{a}$.

The equation which holds for the motion of the hammer during the initial period, $0 \leq t < \frac{2l}{a}$, before the reflected wave arrives at the head, has a simple solution, from which, with the aid of the assumed head-conditions, the displacement-function f representing the down-travelling wave can be determined for that interval. The function F, representing the reflected wave travelling up the pile, can then, from the assumption as to the character of the foot-resistance, be expressed in terms of the function f for a time $\frac{2l}{a}$ earlier. Hence, since f is known for the interval $0 \leq t < \frac{2l}{a}$, F will be known for the interval $\frac{2l}{a} \leq t < \frac{4l}{a}$. This procedure, applied to successive intervals $\frac{2l}{a}$, determines all waves travelling up and down the pile at any selected moment. By combining the waves at any point the displacement, and therefore the stress, may be found at any desired time t.

The theory leads to the conclusions that: —

1) The distribution of stress along a pile at a particular moment is, in general, not uniform.
2) The maximum stress at every point in the pile increases with increased stiffness of cushion.
3) The maximum stress at the head is proportional to the square root of the height of fall of the hammer.
4) For long piles, or for short ones with light hammers and stiff cushions, the maximum stress at the head is dependent only on the conditions at the head and is the same for all sets. The necessary condition for this is that the maximum value of the stress-wave travelling down the pile shall be attained before the reflected wave arrives from the foot.
5) The maximum stress at the foot depends upon the ground-resistance, being zero if the pile is unrestrained, or attaining a value twice that at the head if no movement is possible. In the first case a wave of tension, and in the second case a wave of compression is reflected from the foot.
6) In favourable circumstances, longitudinal vibrations are likely to be set up, in the same way as a musical note is evoked by striking the end of a steel bar. Theoretically, considerable tensions are possible, attaining a maximum at the midpoint of the length. They are likely to be of short duration, and to be followed immediately by compression.

The Head-Cushion.

The great influence of the head-cushion (the dolly, helmet and packing) in determining the stresses in the pile was shown early in the investigation. If no cushion were present between the hammer and the pile-head the stress at the head would rise almost instantaneously to a maximum. The head-cushion decreases both the rate of increase of stress and its maximum value. Stresses throughout the pile are similarly affected.

The stiffness of the head-cushion, denoted by k/A, is a factor to which reference will be made frequently. For an assumed head-cushion in which stress is proportional to strain, k is the usual stiffness-constant as applied to springs and is equal to the force required to produce unit compression, that is, $k = E'A'/l'$, where E' denotes Young's modulus of the dolly, A' its cross-sectional area and l' its length. If A denotes the area of the pile-head, k/A is, therefore, the stress on the pile-head required to produce unit compression, and is equal to Young's modulus divided by the length or thickness of the material, when the cushion is of the same cross-sectional area as the pile. It varies inversely as the thickness and is independent of the stress. The stiffness-constant for a hardwood dolly with a reasonably constant Young's modulus is obtained with sufficient accuracy from this expression. Helmet-packings, however, exhibit a non-linear relationship between stress and compression, and the appropriate value of $\frac{k}{A}$ depends upon the magnitude of the imposed stress. It is therefore necessary to specify $\frac{k}{A}$ as "at ... pounds per square inch"[2].

If the dolly and packing have effective constants $\frac{k_1}{A}$ and $\frac{k_2}{A}$ then $\frac{k}{A}$ for the combination is given by

$$\frac{A}{k} = \frac{A}{k_1} + \frac{A}{k_2} + \dots \frac{A}{k_n}$$

The cushioning effect is chiefly due to the packing beneath the helmet. The effect of the dolly is small, except in cases where the packing has been consolidated to such an extent that its stiffness has become very high, or where the dolly has become soft by brooming. The stiffness-constants $\frac{k}{A}$ of several types of head-packing have been deduced from the stresses recorded during the driving of test-piles and the results show that the value of $\frac{k}{A}$ for the packings used in practice may lie anywhere between 1,000 and 50,000 pounds per square inch per inch. Values as low as 1,000 pounds per square inch per inch. however, only apply to the first blows with new packing, and for practical purposes upper and lower limits of 10,000 and 50,000 pounds per square inch per inch may be used. The effect of the dolly is to reduce these values to about 9,500 and 40,000 pounds per square inch per inch respectively.

During the course of the investigation no form of packing was encountered which answered completely to theoretical requirements. These requirements are: —

1) Low stiffness, as represented by the factor $\frac{k}{A}$.
2) No increase of stiffness during driving.
3) Low cost in relation to durability.

Foot Conditions.

It has been stated that the assumption has been made in the theory that the foot resistance is elastic. In practice the only condition of elastic resistance is obtained when the set, as ordinarily measured, is zero. In all other cases the set may be divided into two portions; set, as ordinarily measured, which will be termed "plastic" set and the elastic movement of the earth, or "elastic" set. The set used in calculations has been designated the "equivalent elastic set" and has been taken as equal to twice the "plastic" plus the "elastic" set. The assumption that the work done at the foot, for the same maximum stress, is independent of the relative amounts of plastic and elastic set is implied. This assumption was tested theoretically by evaluating particular cases for a purely plastic foot

[2] For a fuller explanation of k/A see the Journal of the Institution of Civil Engineers,

and comparing the results with those obtained for a purely elastic foot giving a set equal to twice the set in the plastic case. The results of the comparison were in good agreement.

A typical figure for the estimation of maximum foot stresses is shown in Fig. 3. The complete series of figures covers a range of values of $\frac{kl}{EA}$ from 0.1 to 2.0. These figures furnish an upper limit to the foot stresses in cases of hard driving.

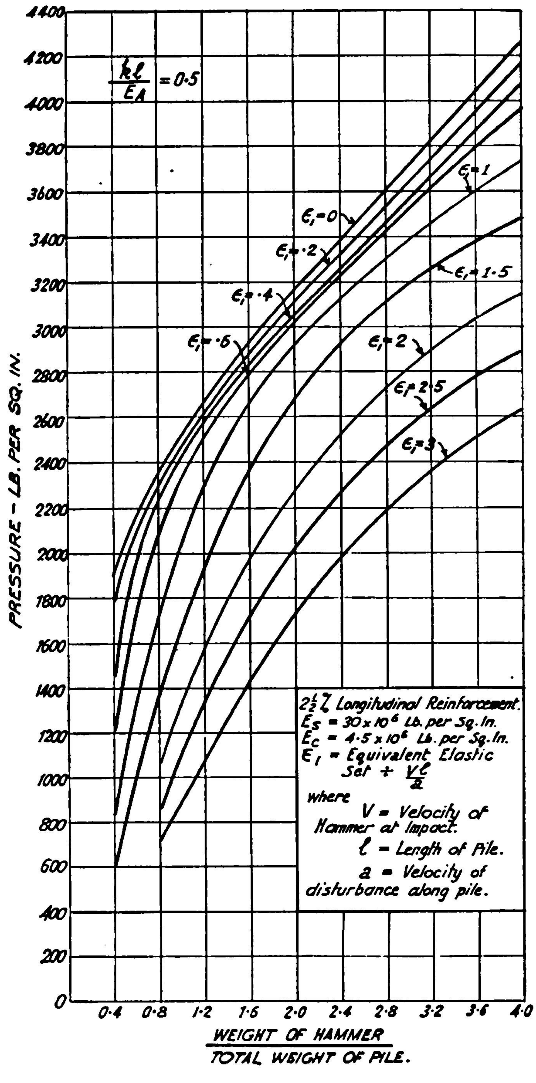

Fig. 3.
Maximum foot pressures for one foot drop.
For a drop of h feet multiply by $\sqrt{h}$).

General Description of Apparatus.

Piezo-Electric Strain-Recorder. The main requirement of a recorder for the short-duration impulses occurring in pile-driving is that all moving parts shall have a very high natural period of vibration, in order to follow accurately and without lag the motion investigated. This requirement led to the choice of a piezo-electric strain-gauge with a cathode-ray oscillograph: the natural frequency of the piezo-electric crystal unit may be made extremely high, so as to take full advantage of the inertia-less response of the oscillograph. Other desirable features possible with such a combination are that the gauges are sufficiently small to be embedded in the pile when cast and, by reason of their simplicity, are inexpensive enough to render their non-recovery from a test-pile of little consequence. The first apparatus constructed was used for tests at the Building Research Station, and has been fully described elsewhere[3]. A transportable outfit was built for the tests under contract conditions, embodying improvements designed mainly to reduce size and weight.

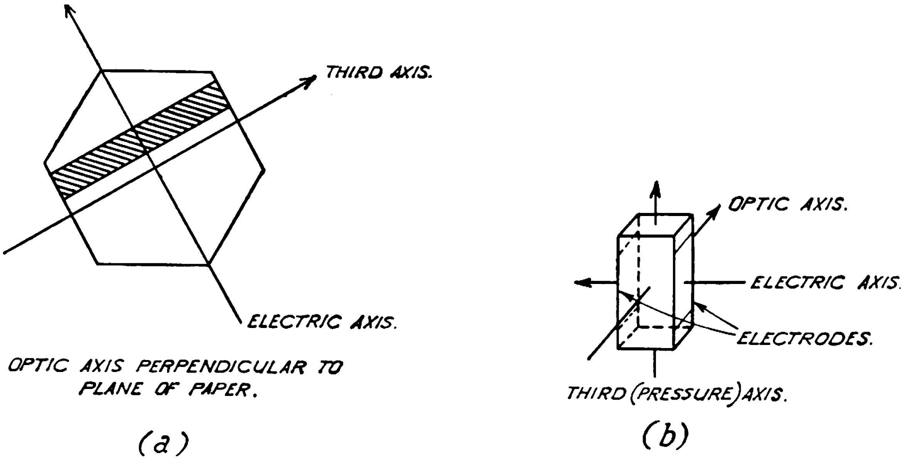

Fig. 4 a and b.

Position of axes in cut Quartz crystals.

The operation of the gauge depends upon the piezo-electric property of quartz, which, in common with certain other crystals, when compressed or elongated along one of the hemihedral axes, develops, at particular regions of the crystal, electric charges which are proportional to the applied force. Crystals, in the form of long rectangular prisms, are cut from the natural material in the direction shown in Figs. 4 a) and 4 b). The prisms, mounted in small watertight chambers, form the gauges which are cast in the pile at selected positions. When in use, the crystals are subjected to pressure along the "third" axis, and electric charges, proportional to the applied pressure, are liberated on electrodes attached to the faces perpendicular to the electric axis. Connection is made to the recording system by highly-insulated leads. A section of a gauge is shown in Fig. 4 c). The numbers denote 1) the quartz crystal, 2) the electrodes, 3) steel plates, 4) stiff spring, 5) heavy circular steel end-plates, 6) thin brass cylinder, 7) and 8) conical seatings, 9) the tube carrying the insulated lead.

When the gauge is assembled an initial load is put on the crystal by screwing up the adjustable seating 7; it then responds to tension as well as compression. In order that the gauge shall be strained to the same extent as the surrounding concrete, its dimensions are so chosen as to make its equivalent stiffness approximately equal to that of the concrete it replaces.

The arrangement of the strain recorder is shown diagrammatically in Fig. 5.

S e t - R e c o r d e r. Sets have been recorded using the method shown in Fig. 6, which has given very satisfactory results. A board carrying a sheet of paper is firmly fixed to the front face of the pile by clamps. A straight-edge, along which

[3] G. Grime, Proc. Phis. Soc., vol. 46 (1934), p. 196.

a pencil is drawn to trace the set record on the paper, is nailed to a heavy baulk of timber, raised from the ground at either end by wooden blocks. The timber forms an effectively fixed base for measurement, since its vibration does not

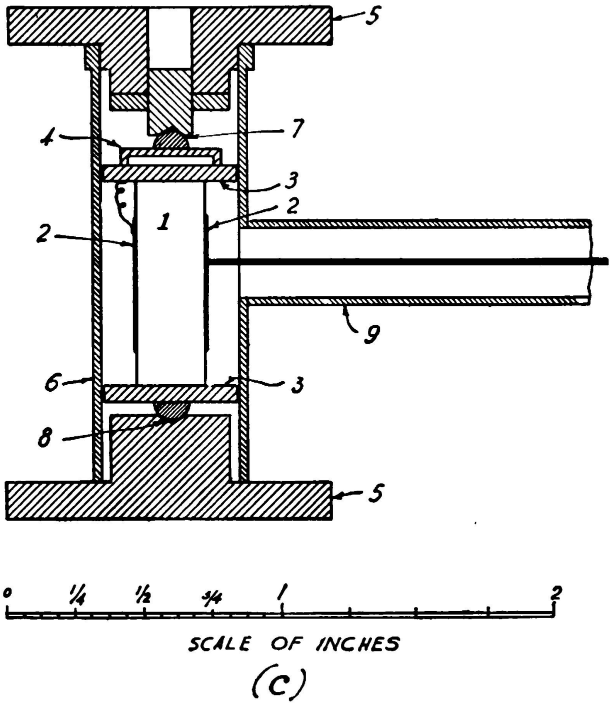

Fig. 4c.
Section of a gauge.

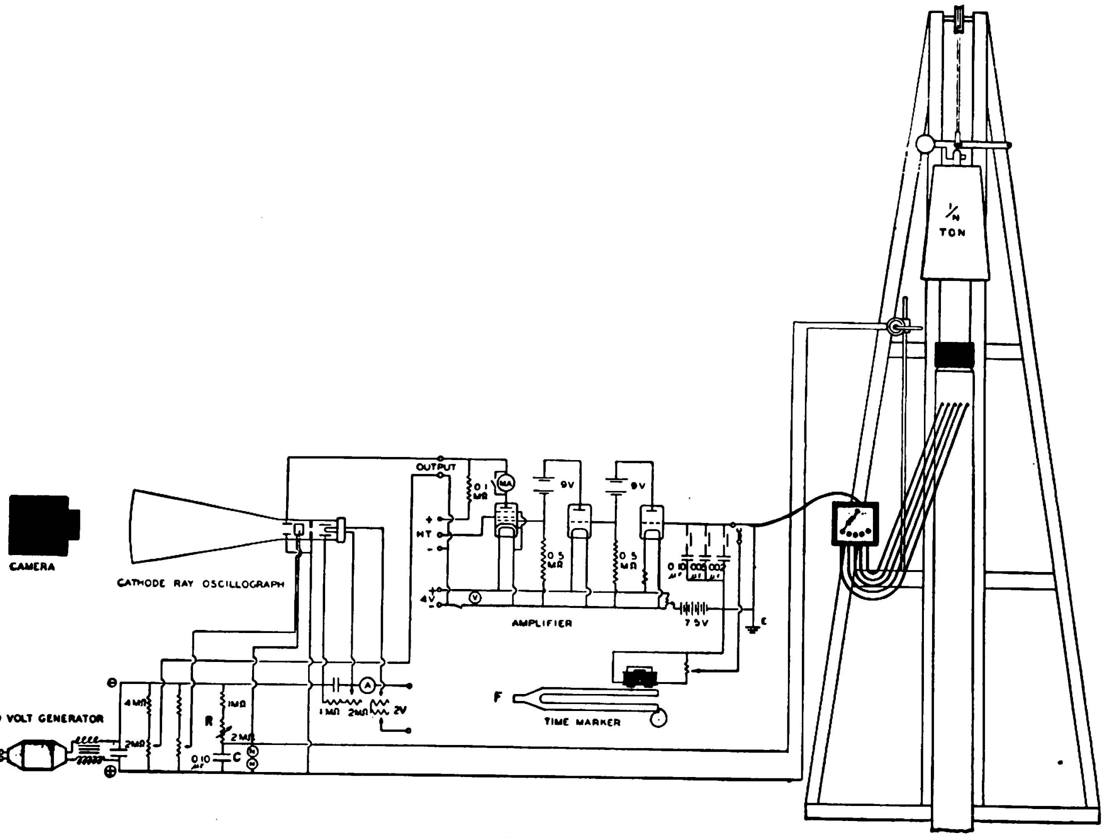

Fig. 5.
Electrical connections of strain-recorder.

become apparent on the record until after the set has been recorded, and is even then of small amplitude. From the record the permanent or plastic set and the elastic or recoverable set or earth-movement can be obtained.

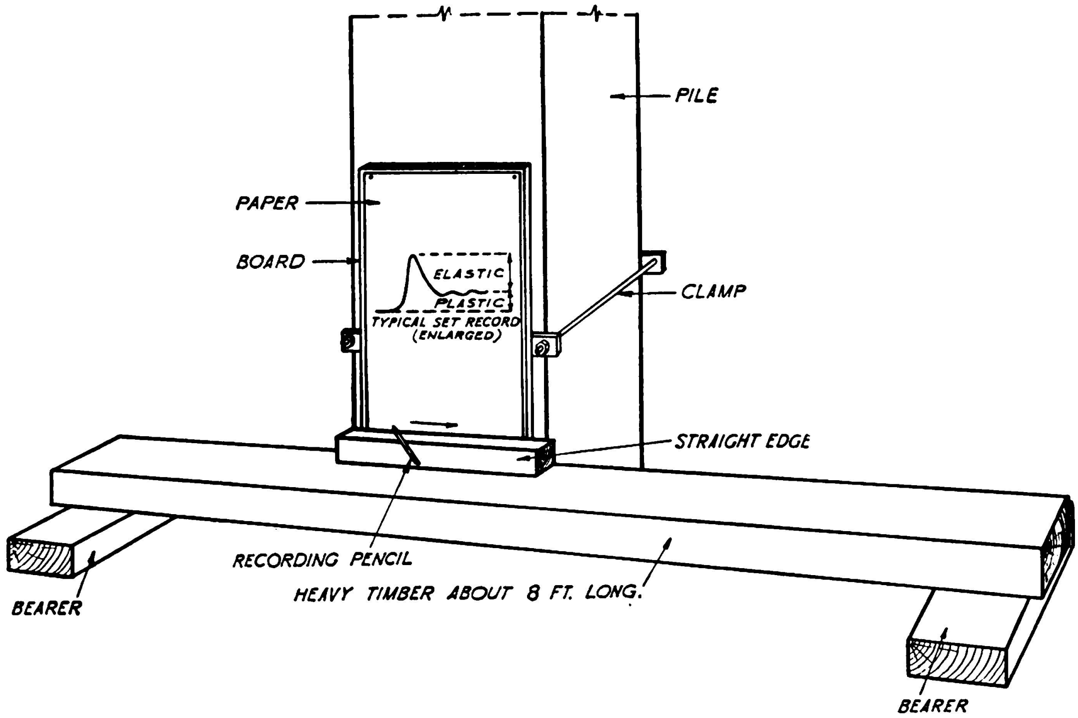

Fig. 6.
Set-recording apparatus.

Peak-Stress Indicator. A simple method of measuring the maximum stress at the head of a pile is provided by the use of a device for indicating when the maximum deceleration of the hammer exceeds a certain preset value. For, if the assumption that the mass of the helmet may be neglected, and the packing at the head regarded as a simple (not necessarily linear) spring, is approximately correct, measurement of the maximum deceleration of the hammer will enable the maximum stress at the head of the pile to be calculated very simply, for the force exerted by the hammer on the head of the pile at any instant = MF, where M denotes the mass of the hammer and F its deceleration.

Maximum-acceleration indicators have previously been used to indicate peak-values of the acceleration of road surfaces subjected to traffic vibrations[4]. The present instrument is somewhat different in construction, and is employed with a new method of visual indication. A mass m (Fig. 7) is held against an insulated stud I by a spring S, the compression of which can be varied by a calibrated screw C. Flat springs F ensure that any motion of the mass m is parallel with the pillars P. The assembly is mounted on the top of the hammer with the axis of the spring S vertical. An electric circuit is completed by the contact

[4] Report of the Permanent International Association of Road Congresses. VIIth Congress, Munich, 1934. (2nd Section: Traffic.)

of the insulated stud I and the mass m, so that when the mass is pulled away from the stud the circuit is broken.

The indicating circuit, which is extremely simple, is also shown in Fig. 7. A dry battery of about 150 volts is connected to three resistances in series. Two of these are bridged by a small neon indicator-lamp, and one of the two

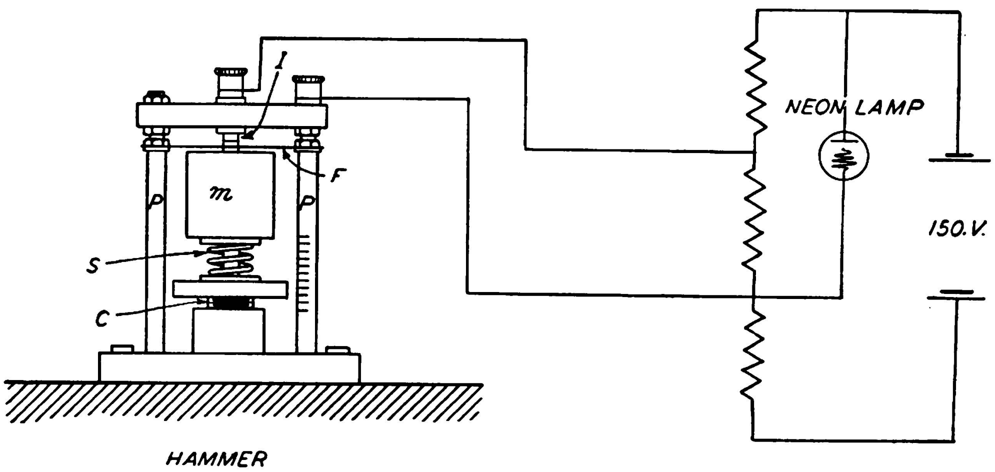

Fig. 7.
Peak stress indicator.

is normally short-circuited by the contact between mass and insulated stud. When the contacts are closed the voltage across the neon lamp lies between the flashing and extinction voltages, so that no current passes. When the contact is broken the voltage across the lamp, now that due to the potential drop across the two resistances, exceeds the flashing voltage and the lamp lights, remaining lit after the contacts close, since the voltage is still at a value higher than that necessary for extinction. This electric circuit is a very sensitive detector of the breaking of contact, and may be made to operate from a break of as little as 0.0002 second duration.

The tests carried out up to the present with the indicator have shown that head-stresses may be measured with a possible error of about 15 per cent.

Experimental Work.

I) Stress-Measurements. Following preliminary tests, which confirmed the general deductions from the wave-theory, experiments were carried out on piles driven into ground, under typically difficult contract conditions.

Details of the piles are given in Figs. 8. The ground conditions are shown in Figs. 9, on which the penetrations at which records were taken are marked.

The piezo-electric recording equipment, housed in a trailer, was operated at a distance of about 20 feet from the pile-driving frame. Connection was made to the gauges by lead-covered leads from 50 to 100 feet long.

The test procedure was essentially as follows for all the piles driven into ground: —

1) A record of the permanent set, averaged for a number of blows, was kept throughout the driving.

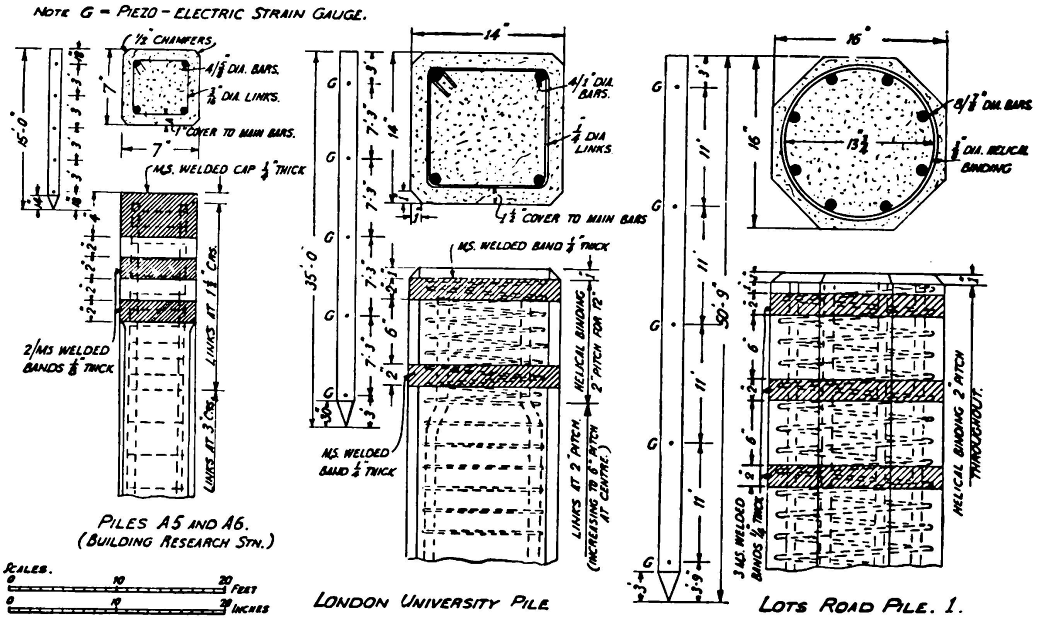

Fig. 8.

Pile details. (Head conditions).

Piles A5 and A6.

Drop-hammers weighing 480, 980 and 200 pounds, with trip release; head-cushion of four, twelve or twenty-four layers of felt initially each $^1/_4$ in thick.

London University Pile.

3 ton single acting steam hammer; 10 in. $\times$ 14 in. $\times$ 14 in. pynkadon dolly, 10 cwt. helmet, and packing of $3^1/_2$ in. of deal on top of 4 layers of sacking.

Lots Road Piles.

3.3 ton winch operated drop hammer; 9 in. cylindrical hickory dolly, 15 in. in diameter, 8 cwt. helmet, and packing of two layers of 2 in. manila rope and eight layers of sacking.

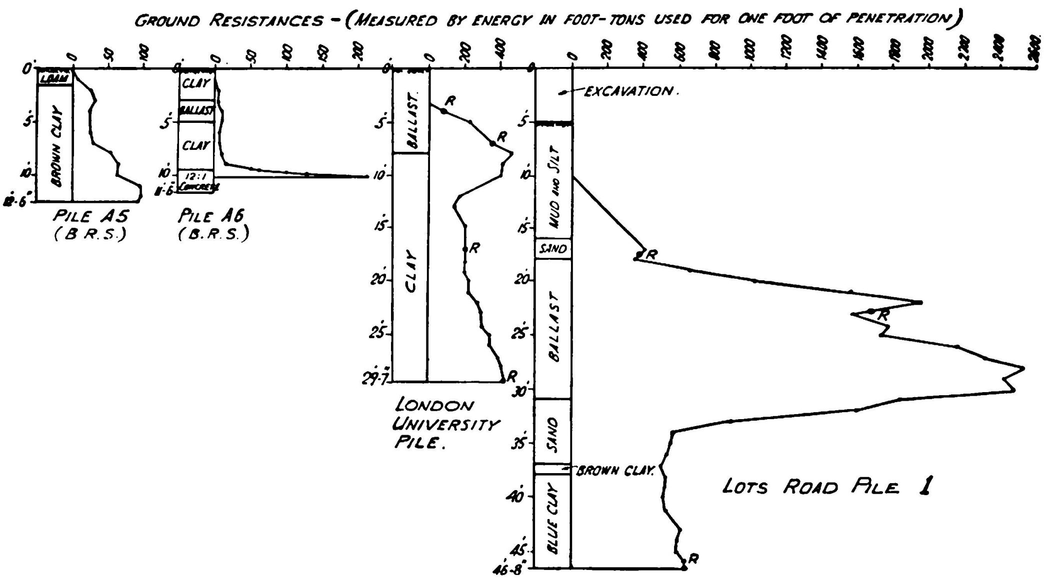

Fig. 9.

Ground conditions.

2) Stresses were recorded for several heights of drop of the hammer at four or five stages of penetration, the positions of which were decided by the ground-conditions (see Figs. 9).
3) Each set of stress-measurements was accompanied by the corresponding records of plastic and elastic set.

Various alterations and re-adjustments of packing material were made during driving, as necessity arose.

Typical records of strains measured with the piezo-electric strain-gauge are shown in Figs. 10, 11, 12 and 13. Their general characteristics agree with

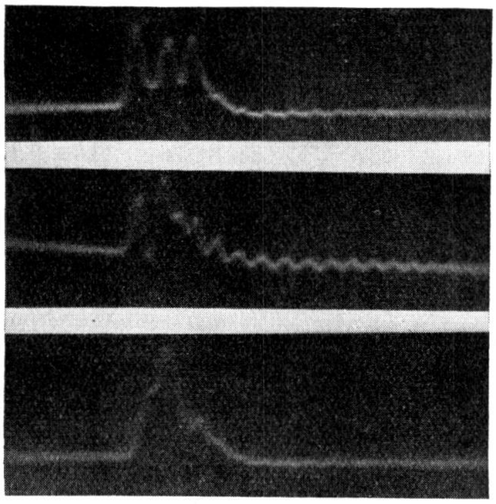

Fig. 10.

Record of the longitudinal vibration of a 15 ft. pile. Vibration frequency 455 per second.

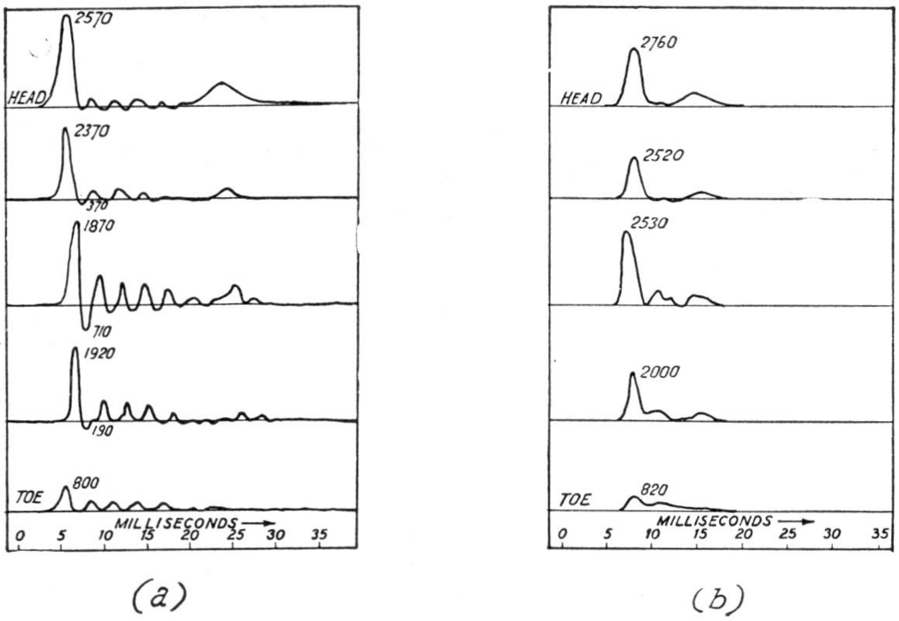

Fig. 11.

Records from a 15 ft. pile driven into stiff clay at the Building Research Station. Driving conditions: — four $^1/_4$ in. felts at head; 980 pound hammer; 24 in. drop. Penetration of point: — (a) 4 ft. 3 in., (b) 10 ft. Set: — (a) 0.55 in., (b) 0.08 in. Figures, indicate peak stresses in pounds per sq. in.

those predicted from the theory. The form of the stress-time curve and the maximum value of the stress vary along the length of the pile, in a manner depending on the ground-conditions. The duration of the record at the head is

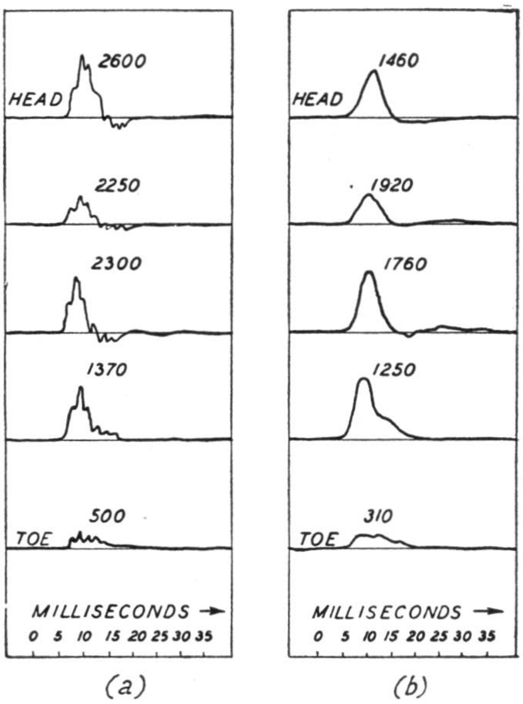

Fig. 12.

London University Pile; Typikal records. Driving conditions: — 3 ton hammer, 24 in. drops, penetration of point 29 ft. 6 in.; (a) contractor's packing with 10 cwt. helmet, (b) twelve felts without helmet. Set: — (a) 0.07 in., (b) 0.04 in. (Figures indicate peak stresses in pounds per sq. in. The low value at the head in (b) is due to back-pressure in the hammer).

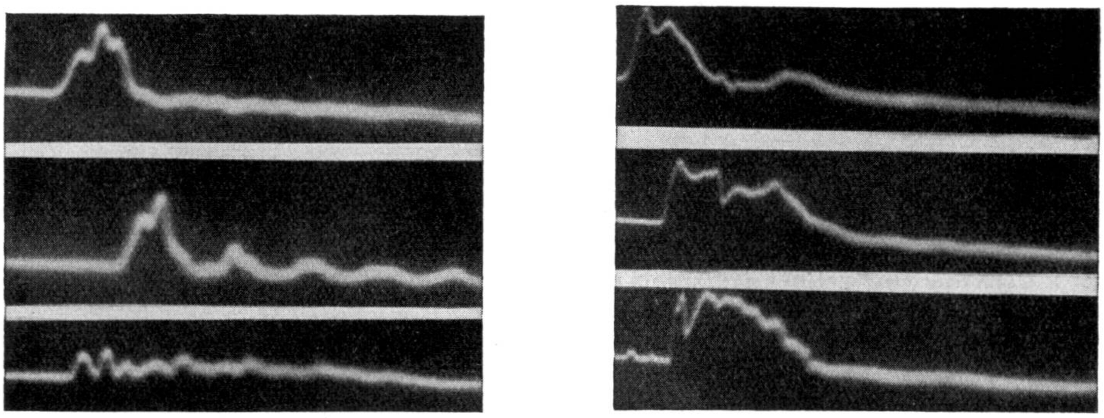

Fig. 13.

Lots Road Pile 2. Typical records for (a) easy and (b) hard driving. Driving conditions: — contractor's packing in 8 cwt. helmet; 3.3 ton hammer; drops (a) 24 in., (b) 36 in.; penetration of point (a) 14 ft., (b) 25 ft. Maximum compressive stresses: — (a) head 1590, middle 1400, foot 520 pounds per sq. in.; (b) head 1930, middle 2170, foot 2760 pounds per sq. in. Sets: — (a) 0.94 in., (b) 0.06 in. Duration of blow at head: — (a) 0.010 second. (b) 0.009 second.

generally of the order of 0.01 second; at the foot it may be greater. The prolonged vibration shown at the middle of the pile in Fig. 10 indicates that, under certain conditions, the pile may be made to vibrate longitudinally at its own natural frequency.

The Effect of Driving Conditions on the Stress in the Pile. In the majority of cases the highest compressive stress induced during driving occurs at the head of the pile. Only when the foot is being forced through an exceptionally hard stratum will it occur at the foot. This is illustrated by Figs. 14

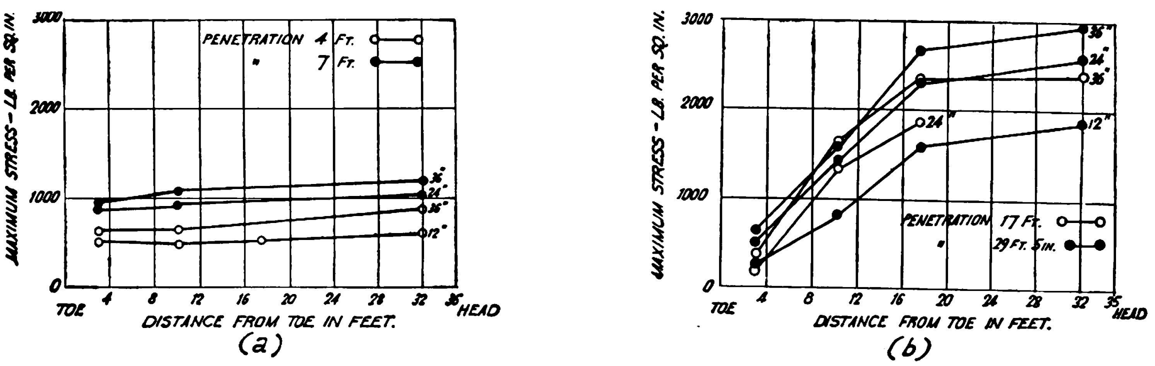

Fig. 14 a and b.

London University Pile.

Distribution of maximum compressive stress along pile at penetrations up to 29 ft. 5 in., with contractor's packing and helmet, and drops of 12, 24 and 36 in.

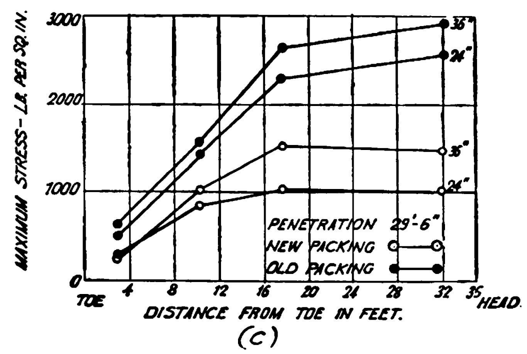

Fig. 14c.

Comparison of new and used contractor's packing for 24 in. and 36 in. drops.

and 15. From theoretical considerations it can be shown that the highest value of the maximum compressive stress must be attained either at the head or the foot, although in certain circumstances values only slightly lower may occur at other positions. At the middle of a pile stresses have been recorded equal to, or even occasionally greater than, those at the head, but these are within the experimental error of the stress-measurements. A comparison of some calculated and recorded head stresses is made in Fig. 16.

From elementary considerations it is apparent that the maximum head-stress for any given set of conditions increases with the weight of the hammer. The increase, however, is for normal head-stresses proportionately less than the increase in weight (see Fig. 16).

In giving an outline of the mathematical theory it has already been mentioned that, in consequence of the finite velocity with which the stress-disturbance, travels, the maximum value of the stress at the head is in most cases independent of the ground-conditions and is determined only by the conditions at the head, that is, by the weight of the hammer, the height of drop, the area of the pile-

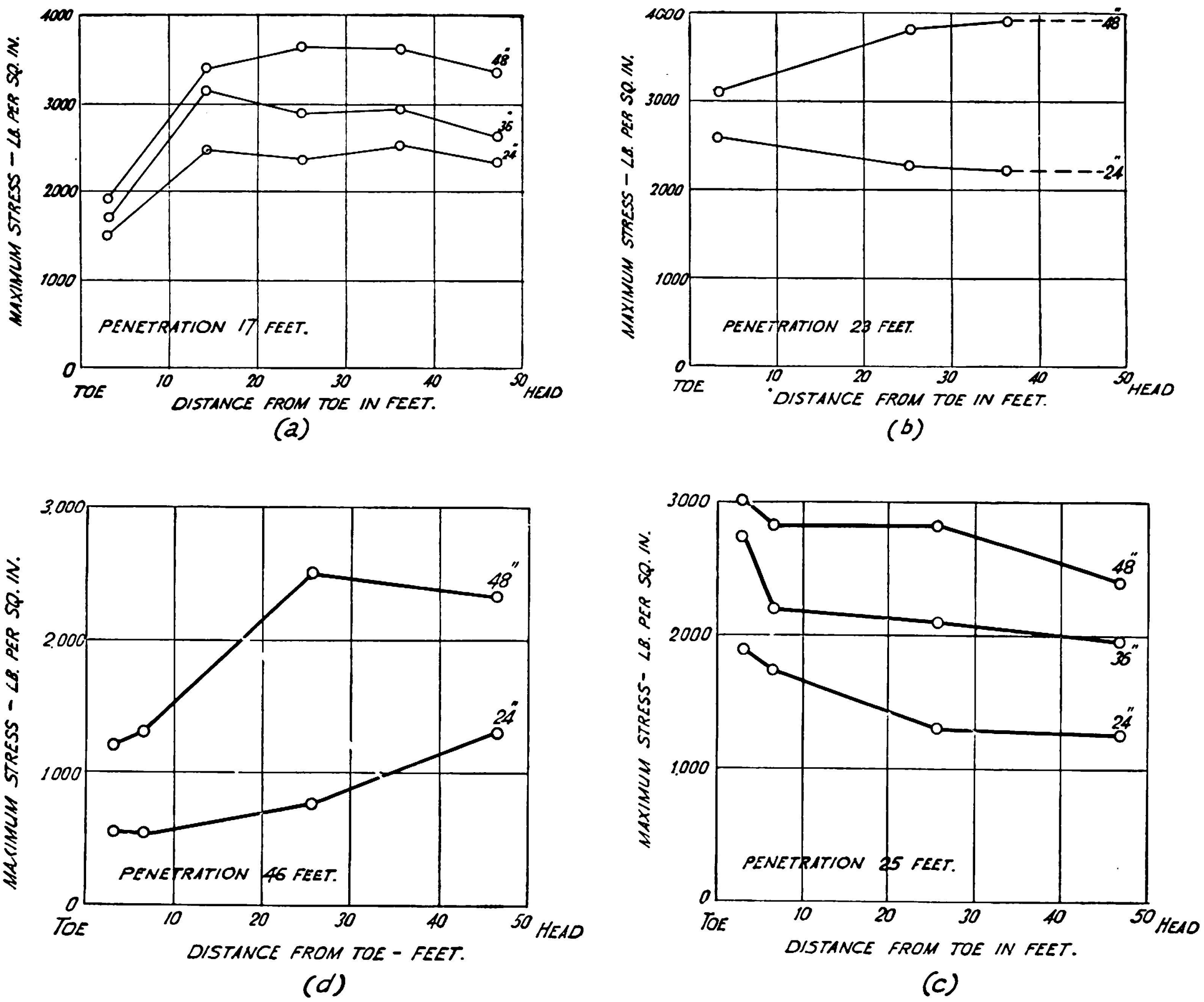

Fig. 15.

Lots Road Piles.

Distribution of maximum compressive stress along the pile at various penetrations, with contractor's packing and helmet, and drops of 24, 36 and 48 in. (a) and (b), pile 1; (c) and (d), pile 2.

head, the physical constants of the pile and the stiffness of the cushion. This is confirmed by experimental results.

A deduction of some importance from the foregoing is that if the packing has initially a high stiffness-constant, the head-stresses in the early or easy stages of driving may be nearly as high as at later stages when the driving is hard.

The advantages of a low stiffness-constant k/A will be dealt with when considering the effect of driving conditions on set. The important effect of an increase of stiffness during driving was demonstrated most convincingly during one of the tests at the London University site, where the softwood packing showed an increase of stiffness which resulted in an increase in the stress at the head of the pile of 100 per cent. (Fig. 14 c).)

Dangerous local concentrations of stress may result from unevenness in placing the packing material on the pile-head. An instance of this was met during the driving of the first Lots Road pile, where the head, which had withstood thousands of blows without sustaining any damage, failed immediately after the insertion of fresh packing. The failure was definitely attributable to the uneven distribution of the packing, which had slipped towards the back of the pile.

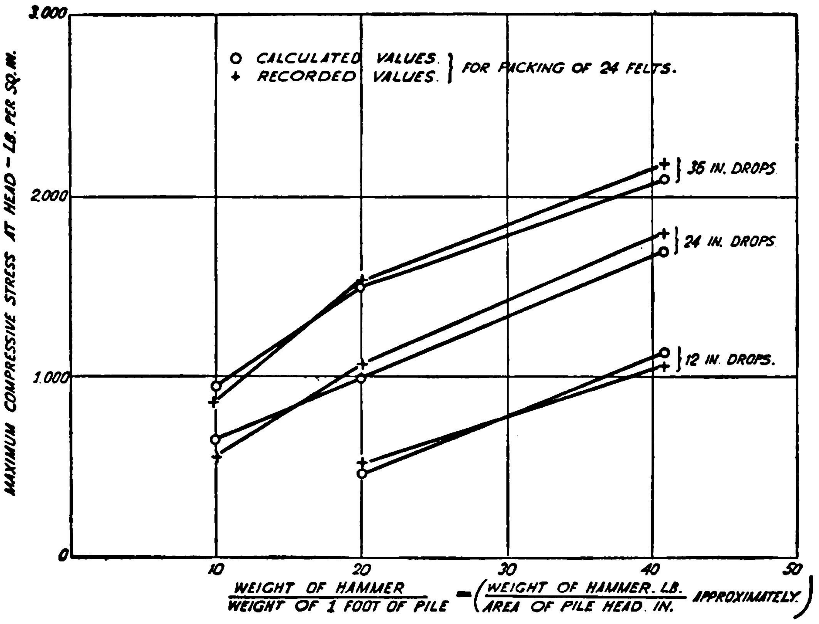

Fig. 16.

Calculated and recorded head stresses for 15 ft. piles.

Experimental evidence (Figs. 12) shows that with packing alone at the head of the pile the stress-time curves, particularly at the head, are smooth in form; with a helmet, the form of the record at the head is that of a vibration of high frequency superimposed on a smooth curve. The helmet may be regarded as a mass supported between two springs, the dolly above and the helmet-packing below. The high frequency appearing in the record is due to the oscillation of the mass on the springs. The magnitude of the maximum stress is in most cases affected little by these oscillations, which are very quickly damped out.

A considerable length of the upper portion of a pile may be subjected to maximum stress almost as great as that at the head. This occurs when no interference from the reflected wave takes place, the only decrease being that due to dissipation of energy by internal and skin friction. The 50-foot Lots Road piles provide examples, the maximum stress along the upper half of the pile remaining approximately constant (Figs. 15). In such cases compressive failure may commence at some distance below the head if a region of weakness exists owing to damage in handling, inadequate transverse reinforcement, or poor concrete.

The stress at the foot depends to a large extent on the set, small sets producing high stresses and vice versa. The foot-stress only becomes important under conditions where its maximum value is likely to equal or exceed that at the head, assuming head and foot to be of equal strength.

The stresses recorded during the driving of test-piles under practical conditions show that only in the case of piles driven on one site, and that one of exceptionally hard driving, were the head-stresses exceeded. (See Fig. 15 c.) Here the foot-resistance was due to a layer of ballast 13 feet thick, and was such as to require two to four hundred blows per foot of penetration.

Calculated and recorded values of the foot stresses are set out in Table I, and in every case the calculated figures are higher by 20 to 30 per cent, than those recorded. It therefore appears probable that the effect of skin-friction and propagation-loss is not negligible, even in such extreme cases of high foot-resistance.

Table I. Comparison of Calculated and Recorded Foot-Stresses.

Pile	Weight of hammer. Pounds	Height of drop. Inches	Packing	Equivalent elastic set. Inches	Maximum foot-stress Pounds per square inch.	
					Calculated	Recorded
Building Research Station (15 ft.)	980	12	12 felts	0.17	1,700	1,300
		24	„	0.39	1,950	1,670
		36	„	0.42	2,860	2,000
	2,000	12	„	0.19	2,440	1,780
		24	„	0.52	2,800	2,230
Lots Road (No. 1)	7.400	24	Contractor's	0.26	3,300	2,600
		48	„	0.45	4,0C0	3,100
(No. 2)		24	„	0.23	2,660	1,900
		36	„	0.34	3,300	2,760
		48	„	0.41	3,950	3,040

Tensile stresses of short duration but considerable magnitude, occurring in the middle portion of the pile, are theoretically possible. This was recognized at an early stage of the investigation and was then considered as a possible major cause of failure below ground. The results of stress-measurements on piles driven under practical conditions, however, do not support this view.

The results show that to set up high tensile stresses the pile must be free to vibrate at its fundamental longitudinal mode, with anti-nodes, that is, places of maximum motion and minimum stress, ot its ends. To fulfil these conditions the ground-resistance must be low and the head-conditions such that the hammer rebounds early and leaves the head free; that is to say, a hard packing and a light hammer must be used. As far as the experimental knowledge goes the evidence is against the occurrence of tensile stresses under practical driving conditions. It is interesting to note that no sign of failure in tension was observed during the driving of the 15 foot test-piles at the Building Research Station, altough tensile stresses greater than the tensile strength of concrete were recorded.

A series of charts given in Figs. 17, 18 & 19 enables any particular piling-conditions to be examined to ascertain whether maximum stresses of 3,000 lb.

per square inch or 2,000 lb. per square inch are likely to be exceeded during driving. Three conditions of head-cushion have been included, namely, soft, medium and hard. For all types of packing tested the hard condition has been found to apply after about 1,000 blows.

From Fig. 17 the ratio of the weight of the hammer and helmet to the weight of one foot of pile is first obtained. From Fig. 18a) or 18b), depending on whether 2,000 or 3,000 lb. per square inch is selected as a maximum for working conditions, the effective height of fall is obtained for the particular conditions of head-cushion required. This effective height of fall is then converted to the

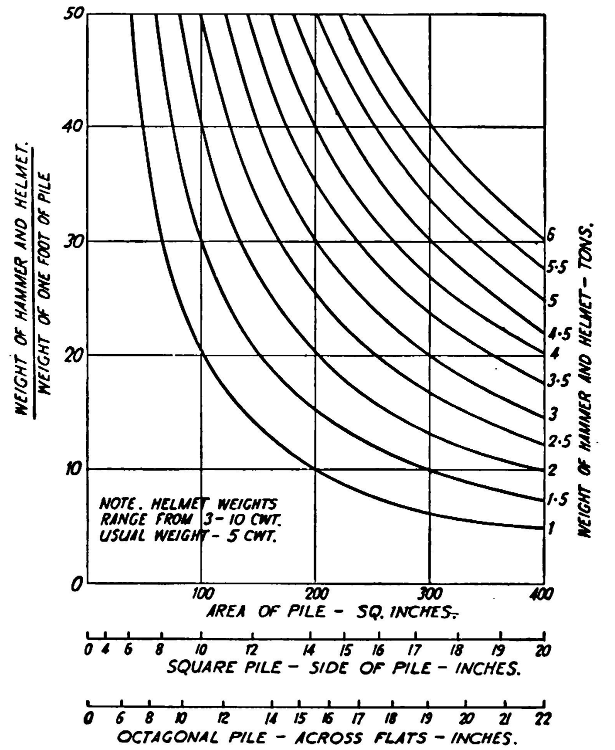

Fig. 17.

Diagram giving the ratio $\frac{\text{weight of hammer and helmet}}{\text{weight of 1 ft. of pile}}$

(Weigth of reinforced concrete taken as 160 pounds per cu. ft.)

height of free fall by means of Fig. 19. Any height of fall greater than this will produce a head-stress greater than that selected.

Figs. 18a) and 18b) also enable the equivalent elastic set which produces a similar stress at the toe, that is, either 2,000 or 3,000 lb. per square inch, to be obtained. Equivalent elastic sets lower in value and falling below the curve produce higher stresses. For foot stresses an allowance of 30 per cent, has been made for friction.

The Effect of Driving Conditions on the Set of the Pile. In the course of the investigation a considerable amount of information has been obtained on the effect of driving conditions on set. It is widely recognized that the energy-efficiency of driving increases with the weight of the hammer used.

The results of the tests are in agreement with this. They also show that the effect is less marked for easy than for hard driving.

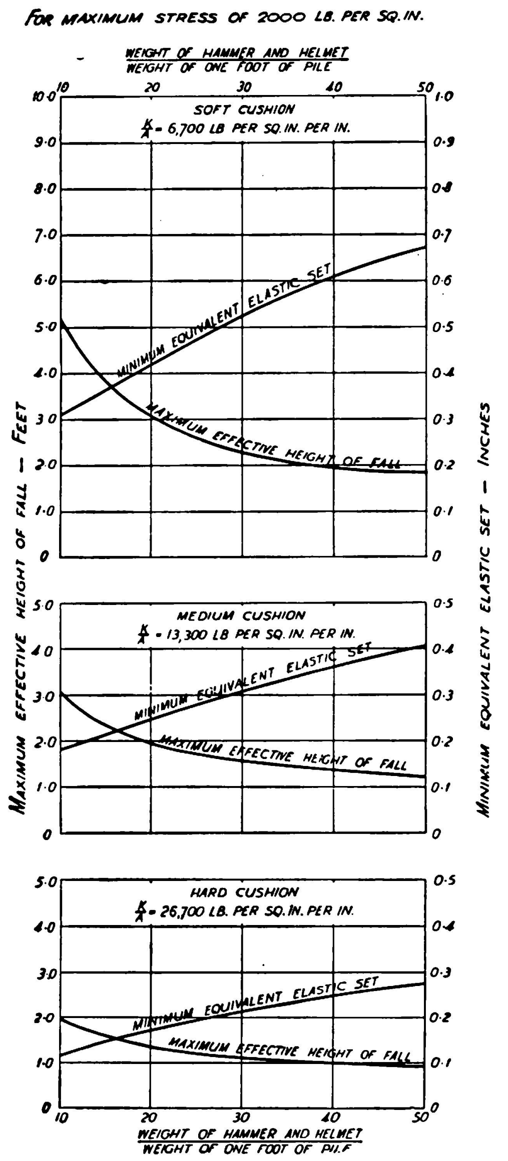

Fig. 18b.

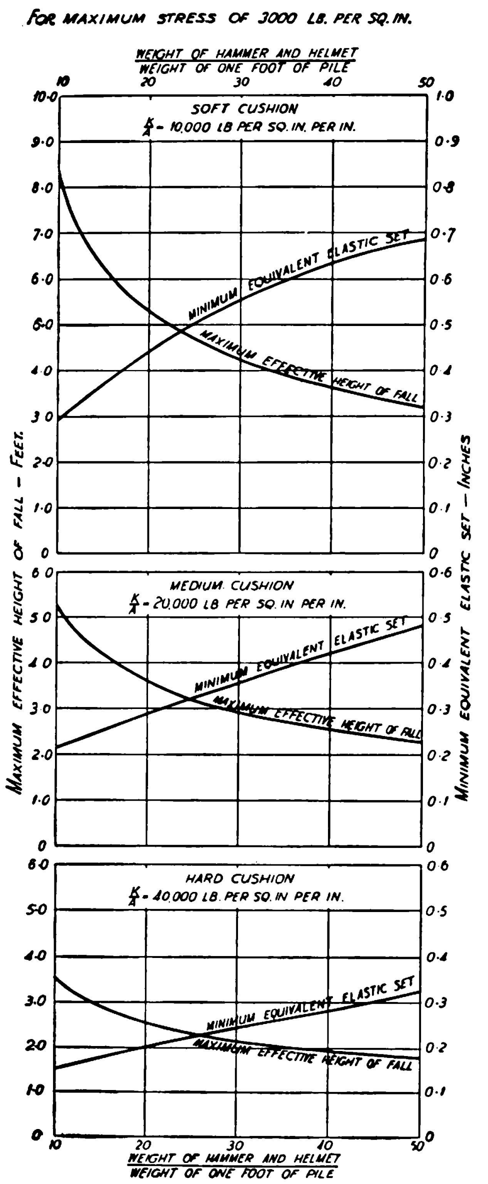

Fig. 18a.

Relation between the ratio $\frac{\text{weigth of hammer and helmet}}{\text{weight of one foot of pile}}$ and the effective heigtht of fall and minimum equivalent elastic set for maximum stress of 3,000 lb. per sq. in. ($2^1/_2\%$ longitudinal reinforcement in pile. Young's modulus for concrete $4.5 \cdot 10^6$ lb. per sq. in.).

The use of a heavy hammer has another advantage at least as important as that of energy-efficiency. Theory and experimental evidence show that, when the heights of drop are adjusted so as to give the same maximum head-stress, the set increases with the weight of the hammer.

The stiffness of the head-cushion has been shown to have an important effect on stress; its effect on set is also considerable, with the important difference, however, that the effect on set is much more dependent on ground-conditions.

Within the range investigated the energy-efficiency of a hammer is found to be greatest when the stiffest packing is used, the gain of efficiency increasing with hardness of driving. For example, the energy-efficiencies of 480 pound and 980 pound hammers used to drive a 15 foot pile through soft clay were practically unaffected by replacing the packing of four $^1/_4$ inch felts by one of twenty-four felts. For moderately hard driving, however, an appreciable decrease of energy-efficiency accompanies a reduction in the stiffness-constant. Again, it was observed that when, during the last stages of the driving of the 50 foot Lots Road piles, test-blows were delivered using thicknesses of twelve and twenty-four felts as packing, the sets for a particular drop were in every instance greater with twelve than with twenty-four felts.

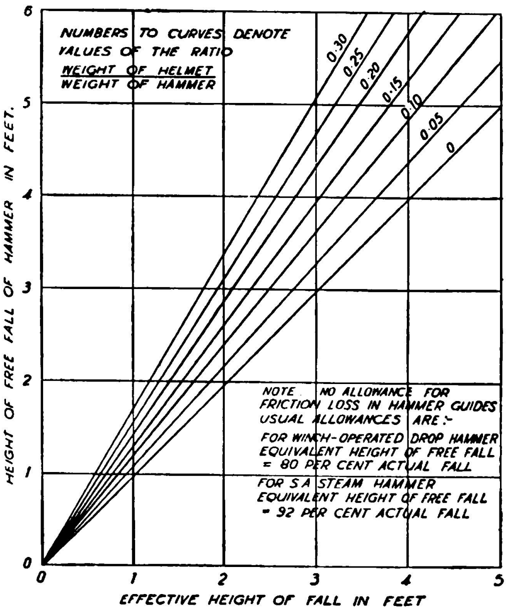

Fig. 19.

Conversion of effective height of fall to height of free fall of the hammer. Numbers to curves denote values of the ratio: $\frac{\text{weight of helmet}}{\text{weight of hammer}}$. Note: No allowance for friction loss in hammer guides usual allowances are: — For winch-operated drop hammer equivalent height of free fall = 80% actual fall. For S. A. steam hammer equivalent height of free fall = 92% actual fall.

This effect is important in connection with the probable accuracy of the bearing capacity of a pile as determined from test-blows. It is widely recognized that no bearing-capacity formula can be expected to be of general application, and that the successful use of any formula is largely dependent on the ability of the user to make appropriate allowance for ground- and driving-conditions. It is obviously of advantage to reduce the number of factors for which allowance has to be made; and since the energy-efficiency of a blow decreases with reduction in the k/A value for all except easy driving conditions, it is of the greatest importance that the packing used for the test-blows should be in as standard a condition as possible. The use of a standardized form of packing for this purpose would obviously be the ideal, but in the light of present information it is difficult to suggest a suitable form in which a packing could be standardized. Failing such a material it would appear better to use well compacted than new packing. Well compacted packing will ensure the maximum set for a given height of drop and does not depend so much on the type of material used as new packing.

The greatest set for a given maximum head-stress is produced by the use of the packing of lowest stiffness. It is clear that this must be so for easy driving, since the energy-efficiency is practically unaffected by packing stiffness whilst the maximum head-stress is dependent on it. Fig. 20 demonstrates the effect

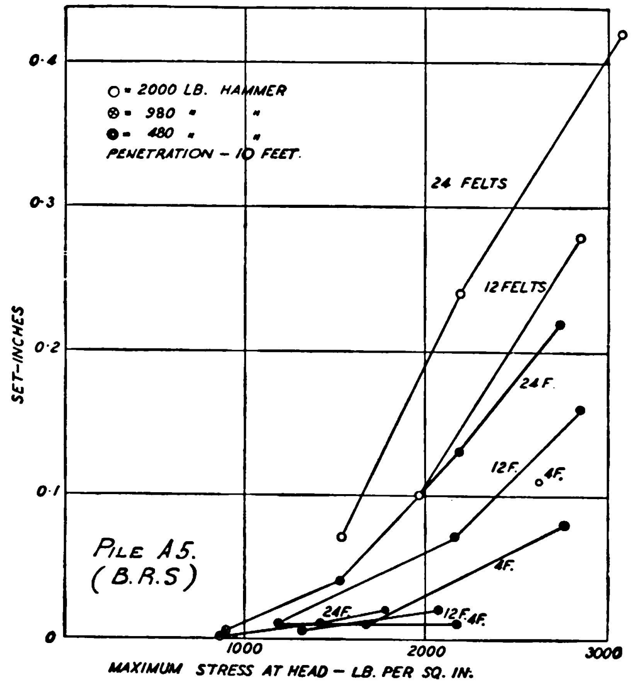

Fig. 20.
Variation of permanent set with maximum head stress for 15 ft. pile.

for moderate driving and, although there is no experimental evidence for hard driving, theory indicates that in this case also the maximum set for a given head-stress is obtained by using the packing of lowest $\frac{k}{A}$ value.

Another observation of considerable importance is that the efficiency of driving, particularly in hard driving, increases with the maximum head-stress attained; in fact with hard driving drops less than a certain height cause no permanent set. During the driving of a 50 foot pile at Lots Road, at the penetration of 41 feet 6 inches the permanent sets for drops of 24, 36 and 48 inches with the same head-cushion were 0.04, 0.10 and 0.19 inch respectively.

It is therefore particularly important in hard driving that for efficient set-production the head of the pile should remain undamaged, as the maximum stress that can be sustained by a damaged pile-head without progressive disintegration is always considerably less than the maximum for a sound head.

The Most Favourable Driving Conditions. The most favourable driving conditions may be defined in two ways: they may be either those conditions which produce the greatest set for the least expenditure of driving energy, irrespective of the stresses induced in the pile; or those conditions which produce the greatest set for the lowest stresses in the pile, irrespective of the energy expended. Since in most cases protection of the pile from failure is the first consideration, and the amount of energy expended, within reasonable limits, is of minor importance, favourable driving conditions will be considered as those enabling a reduction of driving stresses to a minimum.

Within the range of conditions investigated it has been shown that the most favourable conditions of driving, represented by the value of the factor $\frac{\text{set}}{\text{maximum head-stress}}$ occurred without exception when the heaviest hammer was used in conjunction with the head-cushion of lowest stiffness, and that there is reason to suppose that this rule can be applied generally and is virtually independent of the type of ground into which the pile is driven.

Under certain conditions of moderate and hard driving the use of a head-cushion of low stiffness involves a loss of energy-efficiency. In few cases, however, is the loss likely to be of sufficient magnitude to justify the employment of a stiffer cushion, since in most piles the margin of strength would not be large enough to permit an appreciable increase in driving stresses. It appears, therefore, that modifications of driving equipment to obtain better driving conditions should be such as to increase the duration of the blow and reduce the maximum value of the stress.

The Peak Stress Indicator. The instrument may be used for two purposes, fistly to measure the maximum stress at the head of the pile at a particular stage in the driving and, secondly, to indicate, by the lighting of the neon-lamp, when the maximum stress at the head exceeds a certain predetermined value. From the value of the maximum head-stress it is possible to determine 1. the packing constant $\frac{k}{A}$, and 2. an approximate value of the impact-strength of the concrete, by measuring the maximum head-stress at the greatest height of drop that can be used without head-failure.

It is clear that, when set to indicate any maximum head-stress above a predetermined value, the instrument acts as a danger-signal in respect of head-stress, giving warning when the height of drop should be reduced or new packing placed in the helmet.

II) Tests to Destruction. In addition to the tests already described a large number of 15 ft. piles has been tested to destruction. For this purpose the piles were driven against a heavy concrete block using a packing of a suitable type at the foot in order to obtain any desired condition of toe resistance. The piles were 7 in. square and a hammer weighing 980 pounds was used.

The factors investigated were as follows:—

1) Amount and disposition of reinforcement.
2) Type of cement and age at test.
3) Cement content.
4) Curing conditions.
5) Type of aggregate.

A full description of these tests will be found in the papers to which reference has already been made. The conclusions are included in the following section of the paper.

Summary of Conclusions.

1) Stress Conditions. The work, both theoretical and experimental, has shown that the wave-theory of the propagation of stress applies during the driving of reinforced concrete piles. The compression due to the blow travels down from the head and is reflected from the foot as a compression for hard or a tension for easy driving. The stress at any point is the sum of the stresses due to the down- and up-travelling waves. During hard driving compressive stresses may exceed 3,000 lb. per square inch.

The cushion at the head of the pile, that is, the dolly and the packing in the helmet, plays an important part in determining the stresses; the softer the cushion, the lower the maximum stress. For a cushion with a linear stress-strain relation the stiffness-constant (k/A) is the stress on the pile-head to produce unit compression. Cushions usually have a non-linear stress-strain relation and therefore k/A must be defined at "at... lb. per square inch". At 3,000 lb. per square inch values of k/A range, in practice, from 10,000 to 40,000 lb. per square inch per inch and at 2,000 lb. per square inch from 6,670 to 26,700, k/A being approximately proportional to stress. Most forms of packing harden during driving. With piles of length greater than 30 feet the maximum stress at the head is generally independent of the conditions at the foot of the pile.

For very easy driving conditions, that is, with very large sets, the compressive stresses at the toe will be very low and the stress-wave will be reflected as a tension, which when combined with the downcoming compression-wave produces tensions which increase from zero at the toe to a maximum towards the middle of the pile. No failures due to these tensile stresses have been observed. As resistance at the toe increases the compressive stress increases and may theoretically reach twice the value of the maximum head-stress. Values 50 per cent. greater have been recorded.

The foot-stress depends on the total movement of the toe, that is, the set as ordinarily measured and the elastic earth-movement at the toe. For the purpose of stress-estimation the ordinary or permanent set has been termed the "plastic" set and the earth-movement the "elastic" set. When combined, as follows, they have been called the "equivalent elastic set".

Equivalent elastic set

= twice plastic set (or permanent set as ordinarily measured)

+ elastic set (or earth-movement).

The worst conditions at the foot are obtained where the whole of the resistance to penetration is concentrated at this point. Friction at the sides of the pile will have only a small effect on head-stresses but may have an important influence in reducing the stresses below ground.

A simple method of set measurement is satisfactory (see Fig. 6). A correction to the elastic set is necessary to allow for the elastic compression in the pile. This is 0.004 inch per foot of pile embedded where the maximum head-stress is 3,000 lb. per square inch, and 0.003 where the stress is 2,000 lb. per square inch. Further investigation of the order of the elastic and plastic sets occurring in practice is required.

Charts have been prepared enabling the stresses to be deduced for a wide range of conditions.

The best conditions of driving are obtained by using the heaviest possible hammer, together with the softest head-cushion (lowest stiffness k/A), the height of drop being adjusted to give a safe stress. It is suggested that a reasonable minimum value of the ratio weight of hammer/weight of 1 foot of pile would be 30. This gives for 12 inch, 14 inch, 16 inch and 18 inch square piles hammers of $2^1/_4$, 3, $3^3/_4$ and $4^3/_4$ tons respectively (see Fig. 17).

In nearly all cases the equivalent elastic set increases practically in proportion to the hammer-weight, and experimental evidence shows that the plastic set (set as ordinarily measured) increases at a greater rate.

The head-stress may be determined from the peak-stress indicator which is attached to the hammer and measures its deceleration. Alternatively, the instrument may be used to indicate when any predetermined value is exceeded. Measurement of the elastic and plastic sets enables the stress-values thus determined to be used to obtain foot-stresses. Figs. 17, 18 and 19 may be used for this purpose where the indicator is adjusted to 2,000 or 3,000 lb. per square inch.

2) Practical Consideration. To put pile-driving on a proper scientific basis an improved form of head-cushion is required possessing the qualities of permanence and of low and constant stiffness. No entirely satisfactory helmet-packing has yet been discovered, and it is possible that a mechanical device to take the place of the dolly, helmet and packing will afford the most satisfactory solution.

The margin of safety in driving reinforced-concrete piles is frequently so low that slight carelessness in the manufacture and driving of the pile may be sufficient to cause failure. The head of the pile should be carefully formed, and all surfaces in the helmet should be truly plane and at right angles to the axis of the pile. It is most important that the helmet-packing should be placed evenly on the pile-head, and that the layer immediately in contact with the head should be of soft material covering the whole surface. The fall of the hammer should be parallel with the long axis of the pile, and the blow should be delivered as nearly concentrically as possible.

The impact-strength of concrete may be as low as 50 per cent. of the crushing-strength. For a working-stress of 3,000 lb. per square inch a concrete of crushing-strength of not less than 6,000 lb. per square inch is therefore necessary, and for 2,000 lb. per square inch not less than 4,000 lb. per square inch.

To obtain strengths greater than 6,000 lb. per square inch proportions not leaner than $1:1^1/_2:3$ (nominal), that is, 1 cwt. of cement to $1^7/_8$ cubic foot of sand and $3^3/_4$ cubic feet of coarse aggregate, should be used, and the greatest care exercised in the selection of aggregates, the control of water-content, and curing. (It is of interest to note that a crushing-strength of only 3,300 lb. per square inch is required for $1:1^1/_2:3$ High-Grade concrete under the Code of Practice.) For easier driving conditions, where the lower crushing-strength is adequate, that strength might be obtained by careful control with a $1:2:4$ mix.

Curing-conditions have a very marked effect on impact-strength, and piles should be cured under damp conditions as long as practicable. Unless conditions of driving are easy it is recommended that this period should be not less than 14 days. Further information is required on impact-strength and on the factors influencing it.

Longitudinal reinforcement does not affect the impact-strength greatly. The amount of lateral reinforcement, on the other hand, profoundly affects the impact-resistance of a pile, particularly at the head and toe. It is recommended that for a length from the extremities of $2^1/_2$ to 3 times the external diameter of the pile the volume of lateral reinforcement should not be less than 1 per cent. of that of the gross volume of the corresponding length of pile. The diameter of the ties should conform with the usual practice for reinforced concrete, and should be not less than $^3/_{16}$ inch or one-fourth of the diameter of the main bars, whichever is the greater. The minimum spacing of the ties at head and foot should be such as to provide ample facility for placing the concrete. It was observed on an outside contract that the performance of piles reinforced with heavy spirals ($2^1/_4$ per cent.) was definitely good, and although patches of surface spalling occurred, they did not materially affect the resistance of the pile to further driving.

External head-bands placed in the mould before casting the concrete considerably strengthen the pile-head.

The dependence of the set produced on the packing conditions indicates the importance of specifying the condition and nature of the packing to be used in determining the sets on which bearing-capacity is estimated. Failing a standard packing it should at least be specified that the packing shall be well compacted, thus ensuring the maximum set per blow. Up to the present the research has not been concerned with with the bearing-capacity of piles as such.

Summary.

Engineering contractors have frequently experienced great difficulty in complying with specifications demanding that reinforced-concrete piles should be driven through a hard stratum to a set in a lower layer, and in such circumstances many piles have been shattered not only above but below ground. The paper is an abridged account of research carried out at the Building Research Station with the main object of discovering means of estimating when conditions of driving are likely to be destructive to the piles. The work was done with the assistance and collaboration of the Federation of Civil Engineering Contractors.

The research has involved the measurement, by a piezo-electric method, of the strains occurring in the piles during driving. A description is given of the piezo-electric recorder employed. It was found that the strains were profoundly affected not only by the ground conditions, but also by the conditions at the head, and particularly by variations in the amount and state of the head packing.

A mathemathical theory is outlined which enables the stresses to be estimated for certain conditions of head packing and of set. The theory is shown to agree reasonably well with the actual measurements made, not only in small-scale tests but also in tests made under conditions of hard driving on typically difficult sites. The greatest stresses are shown to occur at the head or the foot, and charts are given which enable the stresses at these positions to be estimated.

Conclusions drawn from tests to destruction on 15 ft. $\times$ 7 in. $\times$ 7 in. piles are also given, and recommendations are made for the manufacture and treatment of piles before driving.

VI 4

New Dry-Docks in the Harbours of Genoa and Naples.

Neue Trockendocks in den Häfen von Genua und Neapel.

Nouvelles cales sèches dans les ports de Gênes et Naples.

Professor Ing. G. Krall und Dipl.-Ing. H. Straub,
Rom.

The design of the two large new dry docks in the harbours of Genoa and Naples — the construction of which has since been put in hand, and which will be among the largest and boldest structures of their kind — gave rise to various statical problems which are of general interest.

The transverse dimensions of both works are the same: clear width between side walls 40.0 m depth of dock floor below mean water level 14 m: thickness of side walls 9.0 m — see Figs. 1—3.

The statical behaviour of the two works, however, are fundamentally different, the dock at Genoa being founded on rock while that at Naples rests on a sandy bottom.

A. *Dry dock at Genoa.*

In order to limit the amount of work having to be performed with the aid of compressed air the following working procedure was adopted. The side walls and end wall are built first, together with the connecting groove for the end sill. After the floating caisson gate has been placed in position the dock space is closed on all sides and can be pumped out, allowing the floor to be concreted and the masonry lining to be constructed in the dry.

This method of construction, however, has the disadvantage that pending the construction of the floor, the side walls have to carry the whole of the external water pressure directly on to the rock bottom, since during this period the dock floor is not available to buttress the two side walls against one another as in the finished structure. Since, in the case of the new dry dock at Genoa, the sound rock lies some 5—7 m below the level of the future floor, the loading of the side walls during this place of construction is by far less favourable than in the finished work, a circumstance which gave occasion for the method of calculation described below.

The end wall and side walls are made up of reinforced concrete caissons sunk to impermeable rock of adequate bearing power with the aid of compressed air. For the reasons mentioned above the arrangement of caissons shown in Figs. 4 and 5 has been adopted, alternate caissons being placed lengthwise and

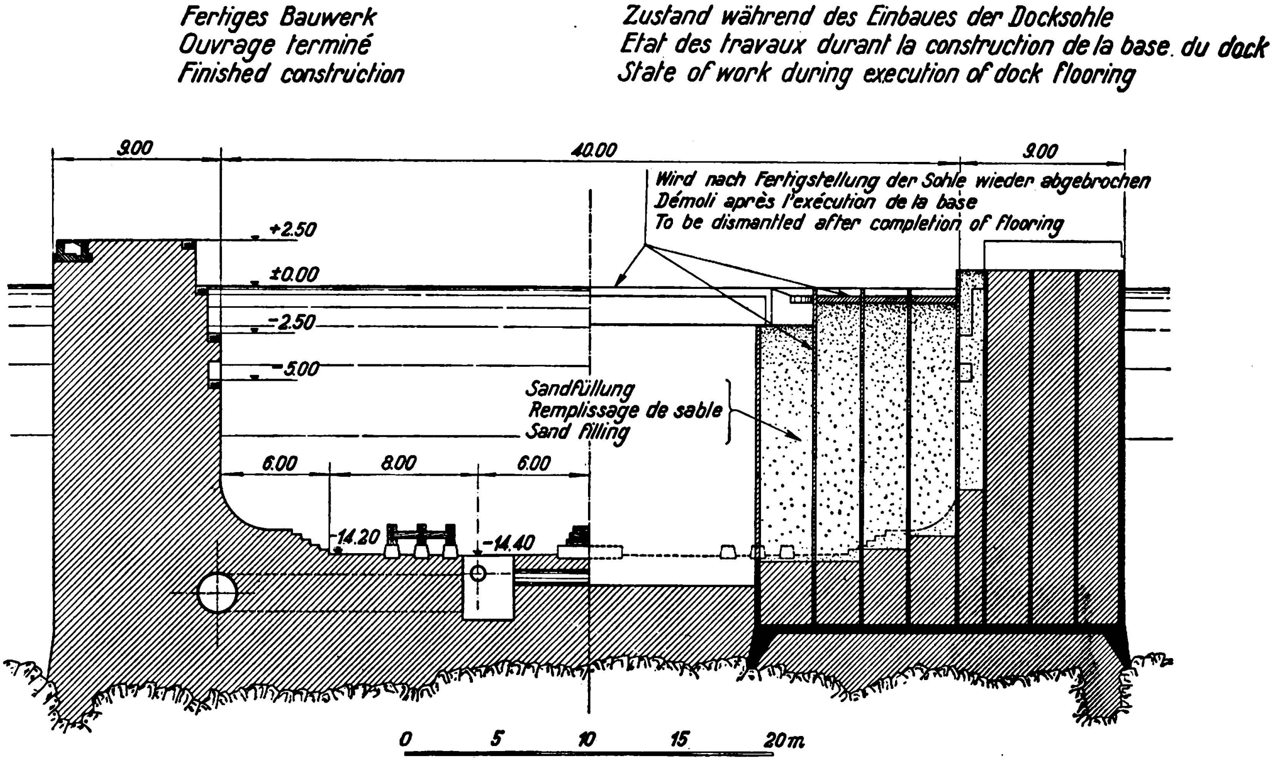

Fig. 1.
Cross section of dry dock in Genoa.

crosswise so that together they form a counterforted dam. In the caissons placed lengthwise, the intermediate walls are so planned that when certain of the cells are filled with stronger concrete they form a horizontal arch. The transverse caissons act as counterforts. In addition each pair of opposite piers is connected above by a strut. After the floor of the dock has been completed these struts and those portions of the counterforts which project inside are cut away.

It ist evident that during the phase of construction now considered, part of the water pressure is borne by the side wall fixed into the rock, while another

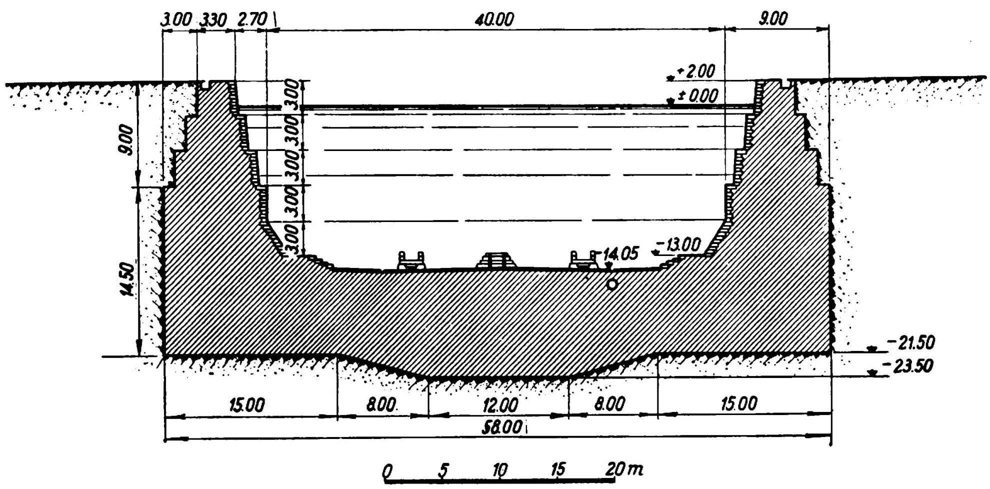

Fig. 2.
Cross section of dry dock in Naples.

part, however, is carried to the counterforts by arch action. In order to determine the stability it is necessary to the probable distribution of the pressure between these two systems[1].

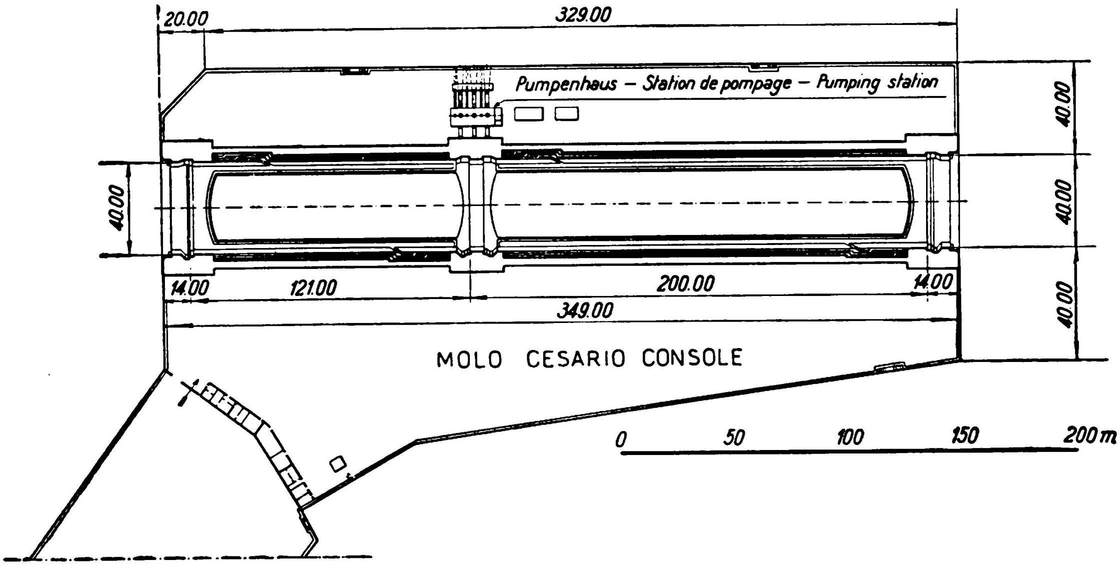

Fig. 3.

Plan of dry dock in Naples.

If p_0 denote the maximum water pressure arising at the point of fixation, the following expression may be given for the share transmitted by the side wall direct on to the rock (see Fig. 4):

$$p'(x, y) = p_0 \left(\frac{x}{h}\right)^m \cdot \frac{1 - \cos\frac{2\pi}{l} \cdot y}{2} \qquad \text{(See Fig. 6)} \tag{1}$$

The approximately sinusoidal form of the surface p in the horizontal direction can be inferred straight away from the mode of action of the dam as here explained, and any such small deviation from this shape as may occur has scarcely any effect on the final result.

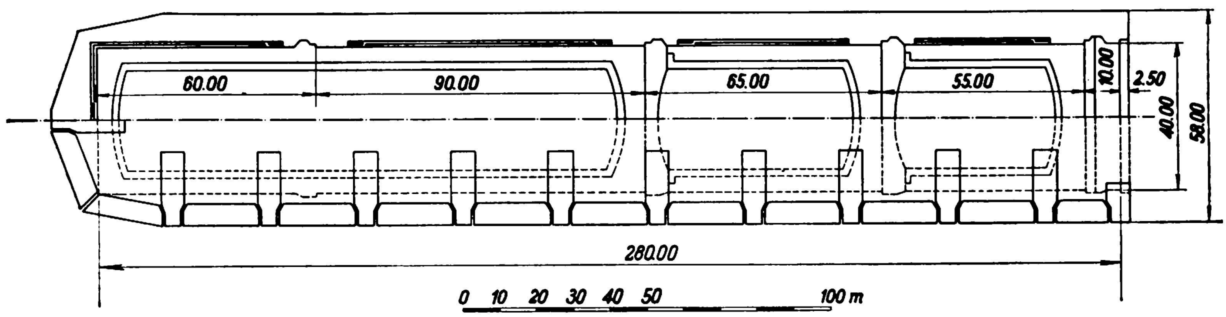

Fig. 4.

Plan of dry dock in Genoa.

[1]) Cf. *K. Krall:* Problemi inerenti alla costruzione del nuovo Bacino di Carenaggio a Genova: Annali dei Lavori Publici 1935, Fasc. II Roma.

The assumption of a parabolic form in the vertical plane enables the problem to be handled mathematically. The nature of the division of pressure between the wall and the counterforts, which governs the conditions of stability, is completely expressed by the magnitude of the exponent m.

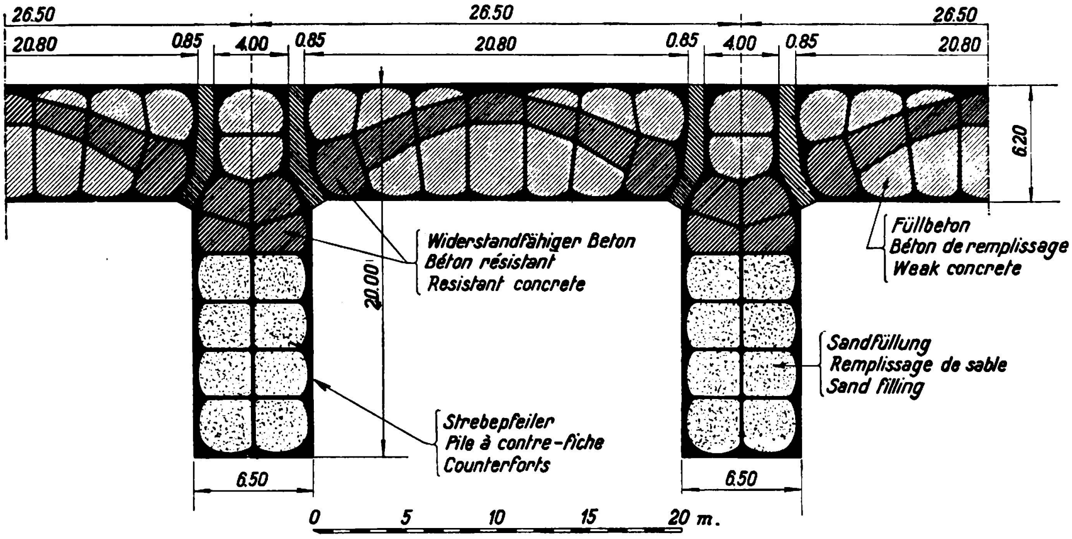

Fig. 5.

Dry dock Genoa: Layout of caissons.

The solution is obtained in the normal way by equating the displacements of the two elastic systems, in the form:

$$\varphi\,(m) = \psi\,[X\,(m);\, m] \tag{2}$$

Where X denotes the force in the upper strut and where the condition is to be fulfilled that equality exists —

I. between half the shortening of the strut, and the displacement of the upper edge of the counterfort,

and II. between the flexure of the central strip of wall which is built in at the foot, and the displacement of the crown of the arch bearing on to the counterforts.

The latter equation must be accurately true for a point at middle height; it may then be shown as a corollary that approximate equality will obtain over the whole height.

To solve the problem mathematically, formulae were next developed for the deflections and elastic lines, the following separate determinations being made:

1) The elastic line for the strip of wall elastically fixed at the foot, under the loading $p_0 \left(\frac{x}{h}\right)^m$.

2) The crown displacement of the arch under the variable load $p_0 \cdot \frac{x}{h} - p'(x, y)$ the supports being assumed invariable.

3) The elastic line of the counterfort built in at its foot, the load thereon being compounded of the bearing reactions of the adjacent arches, the full water pressure over the width of the counterfort itself, and the reaction X in the strut. (Here the counterfort was first assumed to be rigidly fixed at its foot).

4) The additional rotation of the counterfort due to the elastic yield occurring in the cross section built in.

5) The shortening of the strut under the load X.

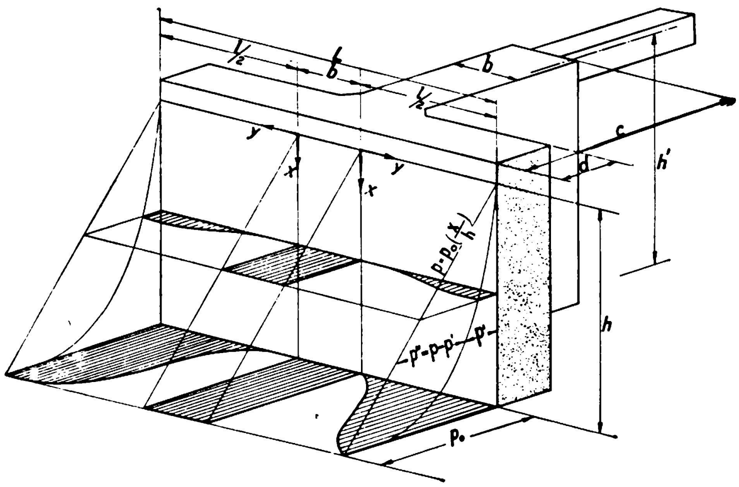

Fig. 6.

Dry dock Genoa: Distribution of water pressure on longitudinal walls and buttresses.

In these expressions the flexures appear as functions of the amount m, the load in the strut X, and, in reference to elastic lines, the ordinate x. The dimensions of the individual constructional members, the moduli of elasticity of the materials, and the ground constant or bedding number C, enter as constants.

If the several displacements are denoted respectively by $w_1, \ldots\ldots, w_5$ in the order indicated above, the critical equations assume the following forms:

$$\frac{1}{2} w_5 = w_3 (h') + w_4 (h') \tag{3}$$

and

$$w_1 (x) = w_2 (x) + w_3 (x) + w_4 (x) \tag{4}$$

When $x = \frac{1}{2} h$ is inserted in (4) as stated above, the only unknowns remaining in these two equations are the quantities m and X, and the solution corresponds with the statement (2) as already given:

$$\varphi (m) = \psi [X (m); m]$$

where

$$\varphi\,(m) = \frac{h^4}{E_m\, i}\left\{\frac{\left[\frac{m}{2}+1+\left(\frac{1}{2}\right)^{m+4}\right]}{(m+1)\,(m+2)\,(m+3)\,(m+4)}+\frac{E_m}{2\,C\,h}\frac{1}{(m+1)\,(m+2)}\right\}$$

$$\Psi\,[X\,(m);\,m] = \frac{l^4\left[1+\left(\frac{f}{a}\right)^2\right]}{64\,f^2\,E_a\cdot s}\left[1-0{,}6\left(\frac{1}{2}\right)^{m-1}\right]$$

$$+\frac{h^4\,L}{120\,E_m\,I}\left\{1{,}50+\left(\frac{1}{2}\right)^4-\frac{1}{2\,L}\cdot\frac{120\left[\frac{m}{2}+1+\left(\frac{1}{2}\right)^{m+4}\right]}{(m+1)\,(m+2)\,(m+3)\,(m+4)}\right.$$

$$\left.-\frac{X}{L\,h}\left[60\,\frac{E_m}{C\,h}\cdot\frac{h'}{h}+40\left(1-\frac{3}{2}\left(\frac{x'}{h'}\right)+\frac{1}{2}\left(\frac{x'}{h'}\right)^3\right)\right]\right\}+\frac{10\,E^m}{C\,h}\left(1-\frac{1}{2\,L}\cdot\frac{6}{(m+1)\,(m+2)}\right)$$

$$X\,(m) = \frac{1}{30}\left(\frac{h}{h'}\right)^2\frac{C\,h}{E_m}\,(L\cdot h)\left\{\frac{1-\frac{1}{2\,L}\cdot\frac{30\left[(m+3)-(m+4)\,\frac{h-h'}{h}\right]}{(m+1)\,(m+2)\,(m+3)\,(m+4)}}{1+\frac{I}{F_p\,h'^2}-\frac{D\,C}{2\,E_p}+\frac{C\,h'}{3\,E_m}}\right.$$

$$\left.+\frac{\frac{5\,E_m\,\frac{h'}{h}}{C\,h}\left(\frac{1}{2\,L}\cdot\frac{6}{(m+1)\,(m+2)}\right)}{1+\frac{I}{F_p\,h_1{}^2}-\frac{D\,C}{2\,E_p}+\frac{C\,h'}{3\,E_m}}\right\}$$

Here E_m = modulus of elasticity of caisson plus concrete filling (mean value)
E_a = modulus of elasticity of horizontal arch
E_p = „ „ „ „ strut
I = moment of inertia of effective cross section of counterfort
i = „ „ „ „ strip of wall of unit length $= \frac{d^3}{12}$
F_p = cross section of strut
s = thickness of arch at crown
f = "rise" of horizontal arch
a $= \frac{l}{2}$
D = length of strut
h' = heigth of axis of strut above built-in cross section of counterfort
x' $= x + (h' - h)$

The numerical calculation is made by taking a number of values of m (m = 2, 3, 4....) and finding first the reaction X (m = 2, 3, 4....) then the functions φ (m = 2, 3, 4....) and ψ (m = 2, 3, 4....). If now, the curves φ (m) and ψ (m) are plotted on a co-ordinate system, the required value m will be determined by their intersection.

In the actual case of the dry dock at Genoa it was found that m = ∼ 6; hence, in accordance with the conditions of execution and the available qualities of concrete, adopted in the different parts of the structure the following constants were adopted for the calculations:

$$\text{Moduli of elasticity} \begin{cases} \text{for the wall built in at foot } E_m = 1.2 \times 10^6 \text{ tons per sq. cm.} \\ \text{for the arch, } E_a = 1.8 \times 10^6 \text{ tons per sq. cm.} \\ \text{for the strut, } E_p = 2.2 \times 10^6 \text{ tons per sq. cm.} \end{cases}$$

For the ground constant C, in order to be on the safe side a considerably lower value was chosen than might be expected to hold good in reality, namely

$$C = 15\,\text{kg/cm}^3 = 1.5 \cdot 10^4\,\text{tm}^3$$

No experimental values for C are available in respect of bearing areas as large as those here considered (approximately 6.6 m $\times$ 20.0 m. If, conversely, C be assumed as proportional to the width of the loaded surface — as is no doubt approximantely correct in the case of a rock foundation[2] — then the adopted value C = 15 corresponds to a value of about 300 kg/cm^2 on a surface of the dimensions usual in experiments (about 1.00 m $\times$ 0.30 m.

Construction.

The period allowed for construction is the relatively short one of $3^1/_2$ years. In fixing this it was, of course, a matter of the greatest importance that it should be possible to prepare the 47 necessary caissons, having the notable dimensions indicated in Fig. 7, in the shortest possible time.

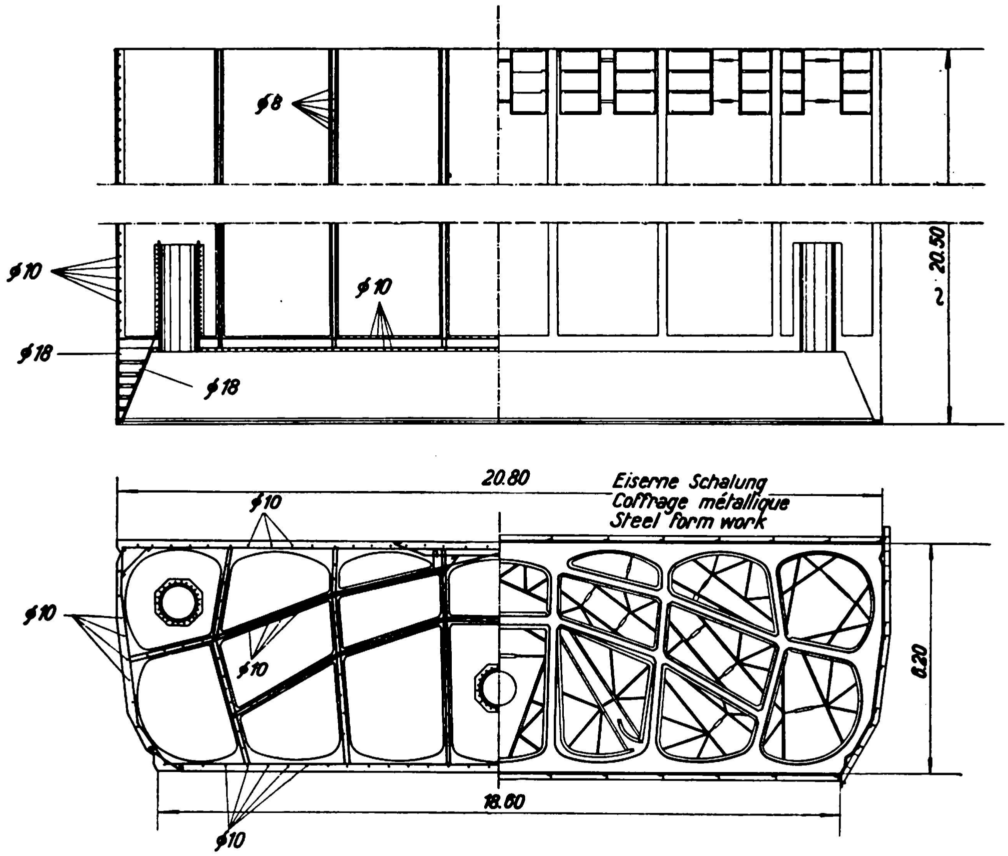

Fig. 7.

Dry dock Genoa: Details of a caisson.

[2]) cf. *Schleicher:* Bauingenieur 1926, p. 931,

For this purpose the joint contractors (SILM-Società Italiana per Lavori Marittimi and SIFC-Società Italiana Finanziaria per Costruzioni) is making use of two small docks specially built for the construction of reinforced concrete floating bodies (Fig. 8). The working chamber and the cells above it to a height of 11.0 m are formed in these, then the floating caissons are towed to their future sites, where the cell walls are continued upward to the necessary height of 18—20 m.

In order to accelerate the work to the utmost and ensure that the concrete surface shall be as smooth and impermeable as possible steel forms are used

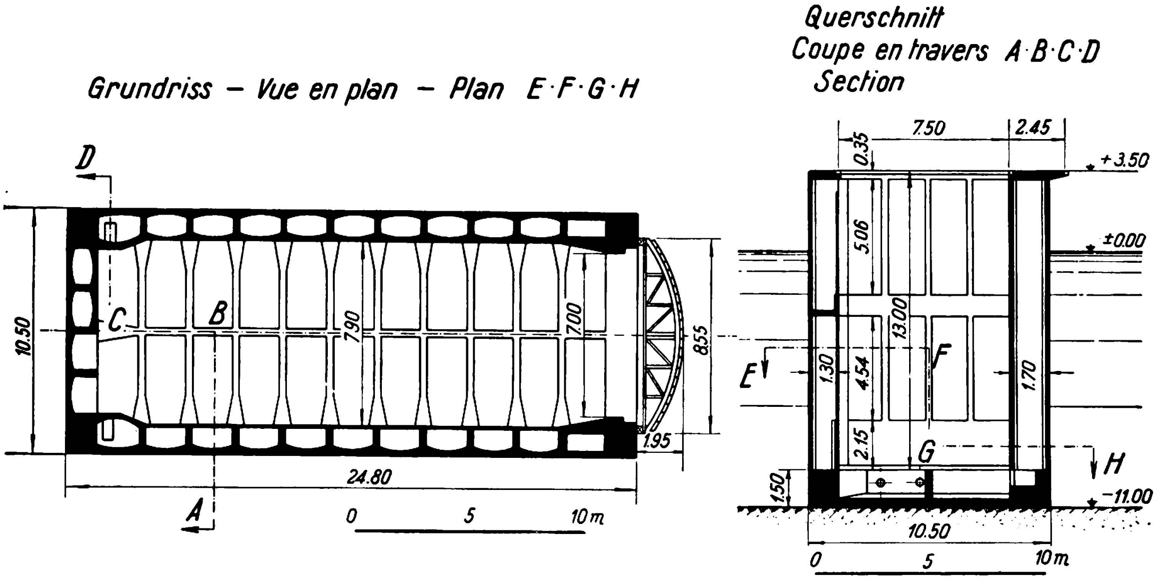

Fig. 8.

Dry dock Genoa: Dock chamber for manufacturing caissons.

(see Fig. 7). Special appliances are adopted whereby the forms may be stripped, raised and refitted within the short period of about six hours, and in this way it is possible to complete one caïsson in each of the two docks in 15—20 days (Fig. 9).

For building the caissons, use is made of a plastic concrete containing 300 kg of cement per m^3 of finished concrete. With a view to greater resistance of the concrete to deleterious effects of the sea water, what is known as pozzolanic cement was chosen instead of ordinary Portland cement. Recently in Italy this material has been much used, with success, for maritime and harbour works; it is a cement poor in lime, and in the course of its manufacture a certain percentage of pozzolanic earth is added to the clinker before grinding, having, like trass, the property of combining with any lime that may be liberated.

The cement adopted has a standard strength of 450 kg/cm^2. Since the conclusion of the experiments made during the first few months to determine the best aggregates and most favourable proportions, it has been possible to obtain with this a briquette strength of 250 kg/cm^2 in concrete 28 days old (Fig. 10).

At the place of sinking, the sea bottom is first dredged down to the rock and is then levelled with the aid of a diving bell suspended from a float; the

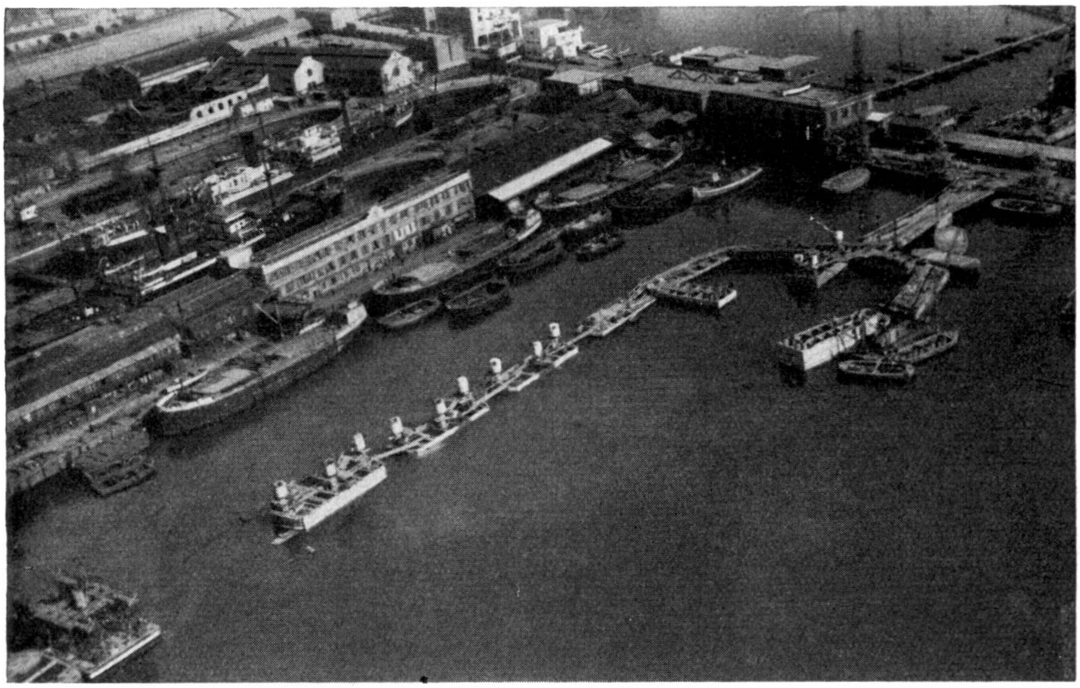

Fig. 9.
Dry dock Genoa: State of work March 1936.

caisson is bedded in its correct position by adding ballast, and is sunk, by the use of compressed air, on to sound rock free absolutely from fissures. The greatest depth reached by any of the caissons sunk up to the present time is — 23.65 m measured from the cutting edge to mean water level. In order to form the waterproofing apron along the foot of the outside of the wall it was necessary to carry the rock excavation a few metres deeper than the cutting edge.

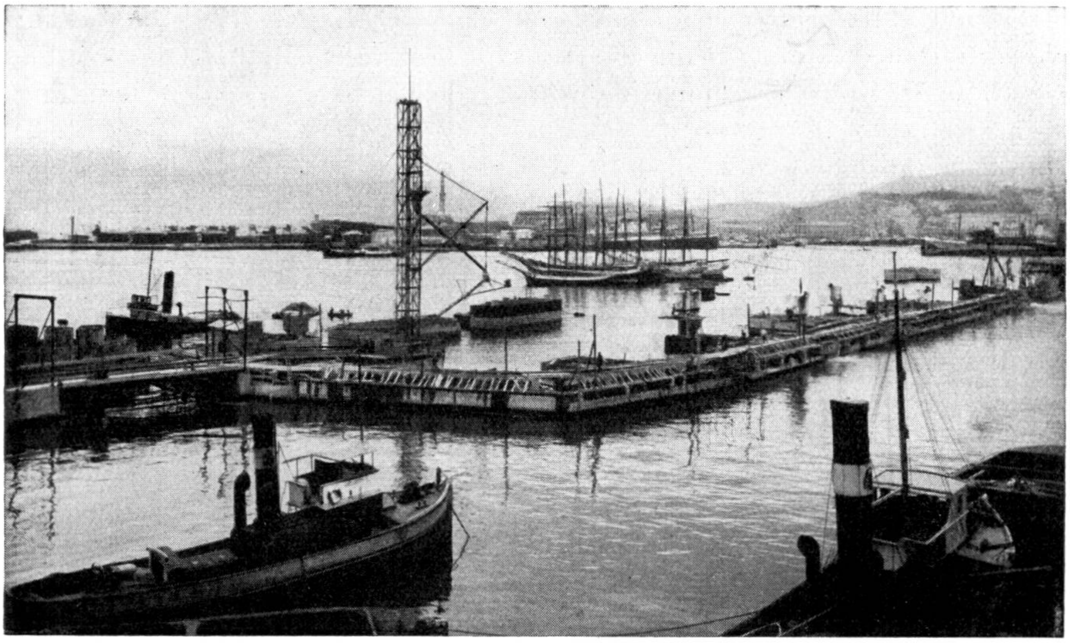

Fig. 10.
Dry dock Genoa: State of work May 1936.

In the case of a few of the pairs of counterforts, the depths actually reached exceed those previously assumed in the calculations by a not inconsiderable amount. In these instances, to restore the conditions underlying the calculations, the opposite counterforts have been buttressed against one another by ground sills in addition to the struts above. These sills are of the same width as the counterforts themselves and their thickness varies according to the depth of the sound bedrock. These again were constructed with the aid of the diving bell mentioned above.

B) Dry dock at Naples.

In contrast to the conditions existing at Genoa, the site for the new dry dock at Naples consists of sand partly interpenetrated by clay, mud and pumice. In broad outline, the method of construction proposed by the contractors (SILM-Società Italiana per Lavori Marittimi) is as follows:

After the bottom has been dredged to the necessary depth two service of reinforced concrete bridges are built outside of and parallel to the side walls of the dock, and two moving steel gantries, each of 68 m span are erected between them to carry the compressed air caissons wherewith the whole of the concrete work under water is done. The latter is formed with a gap separating the floor of the dock from the two side walls so that the settlement of each of these three parts may take place independently. The gap is closed, once more by the use of diving bells, only when no further settlements can be detected.

The cross sections — especially as regards the thickness of the floor — are so dimensioned that tensile stresses cannot arise under any condition of loading; hence steel reinforcements can be dispensed with. The maximum compressive stresses are so low (about 8 kg/cm^2 that instead of cement concrete a mixture of crushed rock lime and pozzolanic earth can be used. Apart from the cheapness of this material — for the most productive sources of pozzolana in Italy are in the immediate neighbourhood of Naples — it offers the advantage of great resistance to the chemical effects of sea water. In the Gulf of Naples there still exist ruins of submarine works constructed with it at the time of the Roman Empire.

In the circumstances described, it was doubly necessary to obtain a correct understanding of the statical behaviour of the structure taking account of the elastic yield both of the material and the foundation. In addition, therefore, to the usual calculation made by the line of pressure method, a further check was carried out in accordance with the procedure briefly described below[3].

For the purpose of analysis the structure is considered as being divided by two planes of section into three parts, as represented diagrammatically in Fig. 11. In this way the portions corresponding to the side walls may be conceived as rigid blocks elastically supported on two sides, while the middle portion may be conceived as an elastic beam. It is required to determine the reactions at the sections (longitudinal and transverse forces and bending moment) complying with the condition that the displacements on each side of the dividing surfaces must balance one another.

[3]) cf. *G. Krall:* Problemi statici delle Costruzioni Marittime Reale Accademia d'Italia, Memorie della classe di sienze fisiche, matematiche e naturali, volume V, 1933.

It becomes evident that the problem can most simply be solved by having recourse to the theory of the ellipse of stress. For this purpose such ellipses are drawn in respect of the two dividing surfaces, one for the side block, one for the central beam. The former being rigid on elastic supports the elements of the ellipse (i. e. its centre, diameter and elastic weight) are determined by the established method (see W. Ritter: Anwendungen der graphischen Statik,

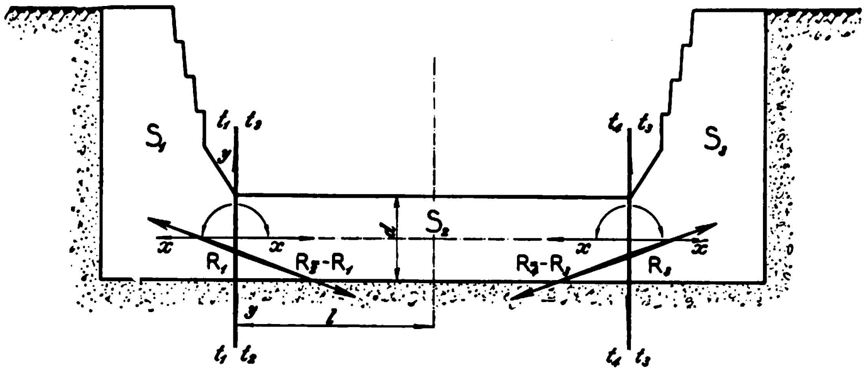

Fig. 11.

Dry dock Naples: Sketch cross section.

4. Teil: Der Bogen, Zürich, 1906 — p. 228). In the case of the beam it is necessary to make use of the theory of the elastically bedded beam. From considerations of symmetry it may be assumed that none but vertical displacements can occur at the central plane of the dock, so the elastic displacements or rotations of the end cross section due to unit longitudinal or transverse stress, or unit moment, are given by the following expressions, wherein the suffix x denotes a longitudinal, and y a transverse force or displacement, and x denotes a moment or rotation.

$$\left.\begin{aligned}
\beta_{xx} &= \frac{1}{EF} \\
\beta_{yx} &= \beta_{xy} = 0 \\
\beta_{zx} &= \beta_{xz} = (0) \\
\beta_{yy} &= \frac{z}{sC} \cdot \rho_3(\lambda) \\
\beta_{zy} &= \beta_{yz} = -\frac{2}{s^2C} \cdot \rho_2(\lambda) \\
\beta_{zz} &= \frac{4}{s^3C}\,\rho_1(\lambda)
\end{aligned}\right\}\ \text{where}\ \left\{\begin{aligned}
s &= \sqrt[4]{\frac{4Ei}{C}} \\
i &= \frac{d^3}{12} \\
\lambda &= \frac{l}{s} \\
\rho_1(\lambda) &= \frac{\mathrm{Ch}\,2\lambda - \cos 2\lambda}{\mathrm{Sh}\,2\lambda + \sin 2\lambda} \\
\rho_2(\lambda) &= \frac{\mathrm{Sh}\,2\lambda - \sin 2\lambda}{\mathrm{Sh}\,2\lambda + \sin 2\lambda} \\
\rho_3(\lambda) &= \frac{\mathrm{Ch}\,2\lambda + \cos 2\lambda}{\mathrm{Sh}\,2\lambda + \sin 2\lambda}
\end{aligned}\right.$$

The significance of d and l may be understood from Fig. 11; C is the ground constant (bedding number); E the modulus of elasticity of the material.

The elements of the ellipse are then given by the following expressions:

$$x_G = -\frac{\beta_{zy}}{\beta_{zz}}\,; \quad y_G = 0\,; \quad g = \beta_{zz}$$

$$i_1^{\,2} = \frac{\beta_{yy}\,\beta_{zz} - \beta_{zy}^{\,2}}{\beta_{zz}^{\,2}}\,; \quad i_2^{\,2} = \frac{1}{E\,d\,\beta_{zz}}$$

The condition that the dividing planes must undergo no mutual displacement can be expressed by constructing a resultant ellipse in which the elastic resistances of the two parts appear as if summed. By analogy with electrical theory, the two ellipses may then be regarded as connected in parallel.

If the elements of the two part-ellipses E_a and E_b are marked by the suffixes a and b, then if the major axes of the two ellipses are parallel to one another (as in the present case) the corresponding elements in the combined ellipse will be determined by the relationships

$$\frac{1}{g_a} + \frac{1}{g_b} = \frac{1}{g} \text{ und } \frac{1}{\lambda_a} + \frac{1}{\lambda_b} = \frac{1}{\lambda} \text{ wenn } \lambda_{ab} = i_1^{\,2}\,g_a, \text{ bezw.} = i_2^{\,2} g_b$$

and the position of the combined ellipse is fixed by the condition that the new major axis will divide the distance separating those of the part — ellipses in the same proportion as the corrsponding values of λ.

If the combined ellipse is established in this way no difficulty will arise in determining the magnitude of the forces taken up by the elastic bedding of the block and the magnitude transferred to the central beam across the section considered, in reference to any given condition of loading or to any given resultant of the external forces acting upon the side block.

The method of construction, as described above, allows settlement of the side walls and the dock floor to take place independently of one another. This being so, it is necessary in the present case to take account only of those external forces which will arise after the two longitudinal gaps have been filled — that is to say the weight of the masonry lining which is to be built after the dock is pumped out, the earth pressure of the back-fill along the sides, and (for the case where the dock space is pumped dry) the lateral water pressure not balanced by internal pressure; also the uplift acting under the floor.

In order to carry out these calculations, the effect due to the external load p_o acting on the floor (or on the elastic central beam) must be replaced by a virtual force P, also vertical, acting on the side block and passing through its centre of elasticity. The magnitude and direction of P are determined by the condition that this force must cause the same movement of the side block in relation to the central beam as is caused by the loading p_0 for which it is substituted; in other words

$$P \cdot i_{1b}^{\,2} \cdot g_b = \frac{-p_o}{C}$$

where i_{1b} and g_b respectively denote the horizontal diameter and elastic weight of the ellipse appertaining to the side block, and C represents the ground constant. P is combined with the external forces which act directly on the side block to give the resultant R.

To divide R between the two components mentioned above, its anti-polar A is determined by reference to the combined ellipse. Then the lines of action of the partial forces appertaining to the side block and the central beam are the anti polars r_a and r_b of the point A by reference to the two partellipses.

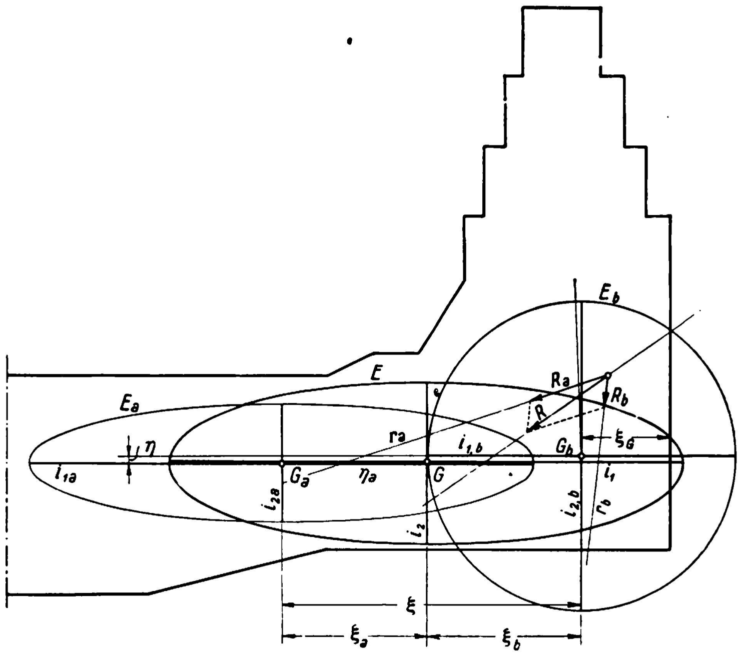

Fig. 12.

Dry dock Naples: Ellipses of elasticity.

The point of intersection of r_a and r_b falls on the line of action of R, which can thus be replaced at once by the two desired component forces, so solving the problem. (See Fig. 12.)

Determination of the ground constant C.

It is clear from the considerations developed above that the ground constant or bedding number C is of outstanding importance. As its value is known to be dependent on the size of the loaded area, and no experimental results are available for such large areas as here, the Società Italiana per Lavori Marittimi who were making the design decided on a large scale experiment. For this purpose use was made of the "Principe di Piemonte" dry dock in Venice, which is of similar dimensions to the new dock in Naples and rests, like the latter, on a sandy bottom.

The elastic movements of the dock, while being repeatedly filled and emptied, were measured at five points with the aid of a telescopic level magnifying 80 times, set up at a suitable distance so as not to participate in the movements of the dock (see Fig. 13). One of the movement diagrams obtained by this means (representative of the others which were all more or less similar) is shown in Fig. 14. The bedding numbers taken from the diagram vary between extreme values of 0.55 and 0.95 kg/cm³.

On the basis of these experiments the value C = 0.75 kg/cm³ = 750 t/m³ was adopted for the statical examination of the dock at Naples.

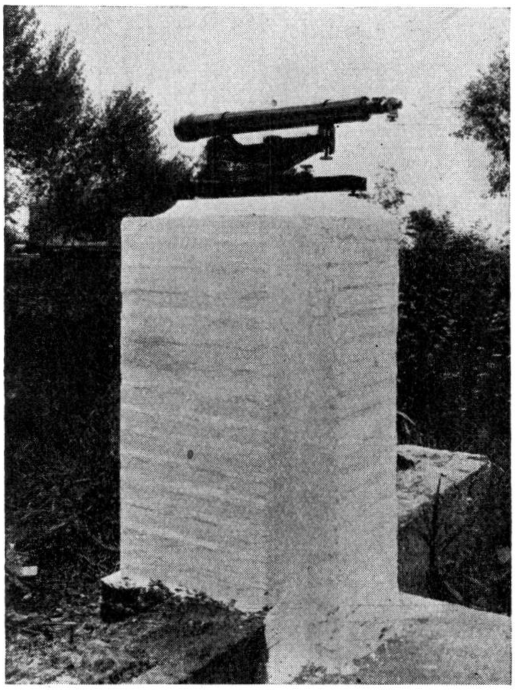

Fig. 13.

Observation station for determining the elastic movements of the dry dock "Principe di Piemonte" in Venice.

In view of the great importance which attaches to knowledge of this ground constant or bedding figure in projecting and designing hydraulic works of large size, such as dry docks, it would be a matter for congratulation if similar experiments were to be carried out elsewhere and their results published.

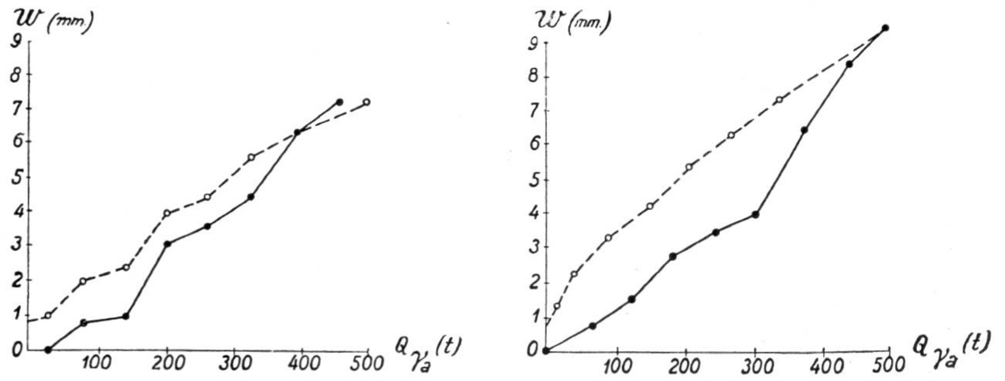

Fig. 14.

Movements of the dry dock "Principe di Piemonte" during filling and subsequent emptying.

Summary.

The Authors describe in their article the recently constructed dry docks of Genoa and Naples. The static behaviour is shown, which is fundamentally different in both cases. The dry dock of Genoa is founded on rock and the dry dock of Naples on sandy ground. To determine the "soil constant" test measurements were taken at the existing dry dock "Principe di Piemonte" in Venice.

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VI 5

Use of Concrete in Dam Construction in Germany.

Beton im deutschen Talsperrenbau.

L'emploi du béton en Allemagne dans la construction des grands barrages.

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1. Concrete dams and retaining walls.

The Dreiläger barrage in the slate hills on the left bank of the Rhine was the first barrage in Germany to be constructed of concrete. It was erected as early as 1909 to 1911. Nevertheless, it can hardly be considered as the precursor of modern concrete barrages because it was made of concrete solely for the reason that no appropriate building stone could be quarried near enough for purchase at an economic figure. In addition, neither in design nor in construction does it possess a single feature of modern concrete structures: highly developed mechanisation, suitable size of broken stone aggregate, high water-cement ratio, density and form of cross section developed in keeping with the special characteristics of the building material. In fact, the wall was constructed of tamped concrete and designed in accordance with principles very commonly used in Germany and originally introduced by Professor *Intze* when devising his Rhenish-Westphalian rubble-stone style of dam construction. The Dreiläger dam is a curved in plan, the water face has a compact rendering with a 0.7 m thick protective rubble stone covering and earth-work at the base, while the upper part of the wall (10 m) consists entirely of rubble stone masonry.

In spite of a few cracks, slight perviousness of the original work joints and signs of suiter formation on the outer face, the dam is in fairly good condition, but, for the reasons outlined above, it has had little influence on the development of German barrage construction.

The Schwarzenbach Dam (Fig. 1), in the northern part of the Black Forest, was started in 1922 and marked the beginning of the construction of modern concrete barrages in Germany. Although there are some excellent granite quarries quite near the site, so that, according to views held until recently, a quarry wall would have seemed the most natural solution, those responsible for the building decided after careful consideration that the dam, which was to be a huge construction for those times, should be of cast concrete. This decision was reached mainly in order to reduce the time required for construction and to meet the difficulty, not to say impossibility, of obtaining at that time — shortly after the Great War — the requisite skilled labour (400 men) and of keeping them for the long building period in a somewhat remote district. Then, too, there was the uncertainty as to the possibility of constructing a really satisfactory watertight quarry wall and, lastly, the necessity for reducing cost.

It is perhaps comprehensible that at that time there was a tendency to maintain the methods introduced by *Intze*, which had proved their value in connection with rubble masonry in Germany, and this led the Board not to take full advantage of the possibilities of this new building material. Consequently, the waterside of the Schwarzenbach Dam was not only made impervious with a gunite rendering and a protection paint coat, but in addition, a further protection lining 0.8 m thick to resist atmospheric and mechanical agents, was applied which again was surfaced with gunite in dovetailed connection with the core wall. These far-reaching and

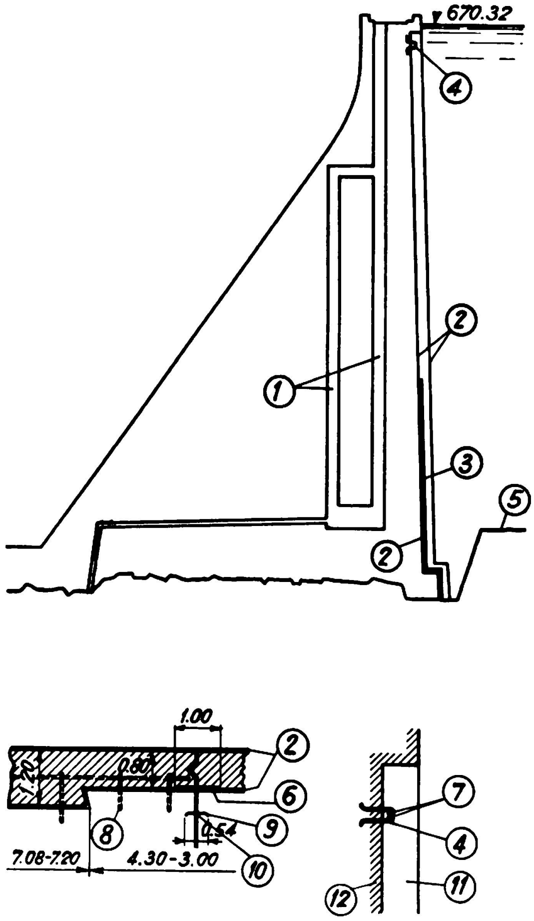

Fig. 1.

Schwarzenbach Barrage.

1) Inspection shaft.
2) Gunite 2.5 cm.
3) Tarred paper insulation.
4) Anchor (stirrups).
5) Rock surface.
6) Three-ply of tarred paper.
7) 2 longitudinal bars 20 mm.
8) Stirrup (at crown).
9) Copper sheet.
10) Tarred rope.
11) Shell.
12) Heart of wall.

really effective methods, which had been introduced by *Intze* and subsequently adopted for the majority of German rubble stone retaining walls with only slight improvements, were applied in still further measure in the lowest third of the Schwarzenbach Dam. Instead of the protective paint coat on the inner surface of the torcrete, a particularly strong insulating layer of three-ply asphaltic cardboard was applied hot. The fact is that the streams of the Black Forest are very soft and sometimes contain free carbonic acid and marsh-acid which give them properties as injurious to cement as those of certain streams in some of the primordinal rock regions of Scandinavia, and it was therefore decided to eliminate this drawback as far as possible in this first attempt of applying concrete to work of this kind. Of course, laying the protective covering and the need for double sluttering for the water face, in addition to the complications of making a really thin protective facing, added to difficulties and to cost.[1]

The facing of the downstream side of the wall also of an expensive character; it consists of a layer 1 m thick of coursed granite masonry, the main object was that of appearance.

Although this wall had been constructed in a hilly district having a high rainfall and much snow (670 m above sea level) it has given the last ten years after being put in commission no repairs worth mentioning. There are no signs of damage on either the upstream or the downstream face. On the water face below the coping wall small deposits of lime, probably due to leakage after heavy showers, affected the outside gunite rendering slightly so that this had to be touched up.

When the reservoir is full there is a total leakage of 1 l/sec from the base and the vertical parts, while for the catchment pipe installed horizontally in the wall near the water face, the figure is 0.4 l/sec. In view of the height and size of the dam this result is very satisfactory.

The *Agger Valley* dam, the first and at present the only cast concrete barrage wall in the Rhineland, the home of *Intze's* rubble walls, represents a great improvement on the Schwarzenbach dam, built only a few years previously. The watertight layer on the water face in this dam was the first example in Germany of the use of facing concrete and there is *no* drainage system at the back. The outer face is lined with "Graywacke"-rubble masonry similar to the Schwarzenbach Dam.

The wall was cast by pouring the concrete through channels from two towers each 76 m high and it has lined expansion joints at equal distances of 30 m. One to two transverse (hair) cracks are visible in each concrete divisional block, particularly those at the ends of the barrage. The facing concrete on the water face has been slightly injured by frost. In the lower inspection passages there are signs of trickling. Signs of moisture are visible on the upper parts of the outer face which disappear in summer and which are probably caused by heavy rains penetrating from the crown of the wall and finding a way down behind the stone facing which was added later to the lined concrete. The maximum drainage of base of the wall (only for a full reservoir) is 1.0 l/sec. Only small quantities of water pass through the expansion joints after having shown originally a few leaks. The total maximum is 3 l/sec for the whole barrage. So far no repairs have had to be carried out.

The comparatively low *Zschopau* Valley dam, at *Kriebstein* in Saxony is the only Saxon retaining wall made by pouring the concrete from towers, although there are many concrete dams in Saxony, built at about the same time. The owners (Saxon Water Supply) considered the matter thoroughly before deciding to build it of cast concrete (as in the case of Schwarzenbach barrage).

The concrete mixture consisted of the following ingredients: 1 of cement: 0.38 trass concrete: 3.89 unusually rich aggregate. This corresponds, exclusive of the trass concrete to 320 kg cement per m^3 of concrete ready for use.

The watertightness of the water face is further increased by a facing of gunite 25 mm thick, rendered smooth and a three coats of inertol, and by drainage arrangement immediately behind this facing. The downstream surface is left unfaced, a new method in German barrage construction.

[1] Cp. Bautechnik 4. VI. 1926.

Apart from surface peeling the wall is in good condition and no repairs have been necessary.

The average annual seepage of the wall, which was very small right from the start, has been diminishing steadily ever since; in 1930 it was 0.14 and in 1935 0.026 l/sec self-caulking effect (Auto-densifying!).

The *Schluchsee* and *Schwarza* dams in the southern part of the Black Forest were built by the same Board (Baden Works), the same contractors (Siemens Construction Company) and partly by the same engineers as the Schwarzenbach dam, and the very different style of construction is a striking example of how opinion had changed in the course of a few years. These latter barrages are not made of liquid, but of "plastic" concrete, which instead of being brought to the site by channels was conveyed by belting and buckets and three rotary tower cranes.

The water face and the downstream surface are treated with facing concrete (0.75 and 1.00 m respectively) and given a further facing 1.50 m thick, in addition the water face has a rendering of torcrete 25 mm thick and a double coat of intertol. At the back there is a drainage system. The expansion joints are spaced at intervals of 12 to 15.5 m in the Schluchsee dam and 10 to 20 m at Schwarza.

In neither case has serious damage been observed on either outer or water faces; surface damage caused by frost and removal of shuttering, however, have warred the appearance of considerable portions of both faces. Most of the expansion joints which extended only to the foot of the service passage became subsequently elongated to the foot of the wall, this increase in length being due to crack formation.

The *Saale Valley* dam near the *Kleine Bleiloch*, built about the same time as the Schluchsee and Schwarza dams, is at present the highest barrage in Germany (70 m.) Unlike these two latter, it is built of cast concrete powred from a stationary concreting bridge, by means of vertical articulated pipes with dumping shovels worked by gravity mixers, to the lower part of which an arrangement of channels on rollers for pouring out the concrete is fixed. Neither the water face nor the downstream surface was treatet with facing concrete or plastering, but the wall was built of a compact mixture of Portland cement, Thurament[2] and crushed aggregate with a liberal admixture of water — about 240 litres per cubic metre of concrete.

The amount of heat developed in the wall, the changes in volume and movement of the blocks were kept under close observation. A number of large cracks formed, and here it is interesting to note that while the spaces between the expansion joints were fairly large — 25 m — four out of eight blocks cracked in the middle and radially even prior to the first filling of the reservoir. They were rendered watertight by dovetail chases cut into the water face, clinker masonry work being added, behind which a mixture of asbestos and bitumen was poured. The result was very good. In spite of this, however, the wall is not perfectly watertight; when the reservoir is full the drainage system on the water face carries a leakage of 25 l/sec. In one of the blocks the middle crack, which was filled up to make it more compact, has cracked further in an upward direction, so that water passes through this crack and flows straight into the drain pipe (16 l/sec when the reservoir is full). It is intended

[2] Thurament is a mortar ingredient composed of ground basic blast furnace slag; its binding power only develops when lime or cement is added. It is produced in Thuringia not far from the building site.

to improve this anti-leakage packing with a view to diminishing the amount of drainage water.

No other damages have been noticed on the outer surfaces of the Bleiloch dam since it was completed in 1932.

The *Zillierbach* dam in the Harz Mountains was not completed until 1935. This means no operating experience is forth-coming as yet, however, an description of this structure is of interest as it is an example of the direction followed in Germany by the development of concrete barrage construction.

The wall is 47 m high and 172 m long, and is the first concrete dam designed in Germany without a curved ground plan. The thickness of the crown — 2.0 m — is remarkably small.

The concrete was made of crushed granite porphyry and diabas rock which had been carefully selected and which were mixed so as to obtain satisfactory granulation (Fig. 2) with 200 kg/m³ cement for the core wall and 300 kg/m³ cement for on the surfaces of the downstream and water faces in thicknesses of 1.0 and 1.2 m respectively. Steel lining was used for both sides.

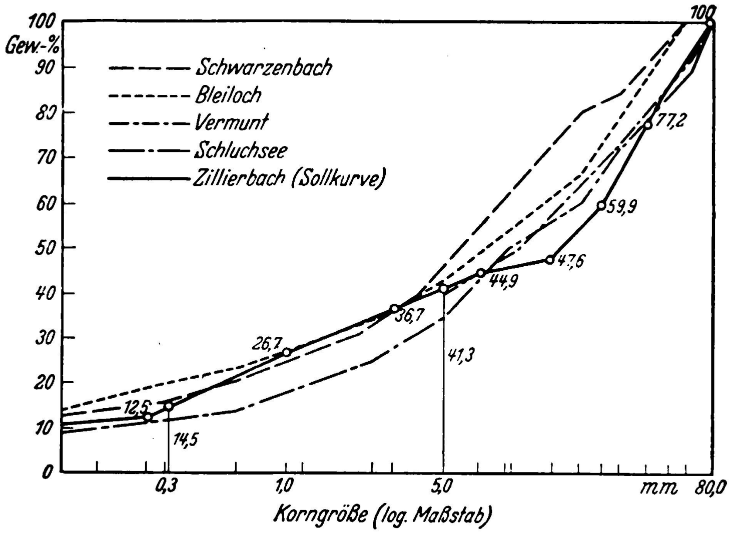

Fig. 2. Comparison of sieving curves (Fuller curves) of recent barrages, including of binding agents (cements).

Korngröße (log. Maßstab) = grain size (log. scale)
Gew. — % = weight in %.
Zillierbach(Sollkurve) = Zillierbach (ideal)

Enough water was added to produce a plastic concrete which could be easily mixed. This meant an addition of 160 to 170 litres of water per m³ of concrete, corresponding to a water cement ratio of 0.8 to 0.85 in the case of the core cement. At first a trass portland cement having a 30 per cent trass content was employed and later on a similar grade of blast furnace concrete. The concrete was conveyed to the moulds by tipping wagons which poured the material into semi-circular gutters and into Y-distribution pieces placed on a steep incline, but most of the concrete (and all the facing concrete) was poured in by in trolleys moved by rotary tower cranes. Inside the moulds the soft concrete was well tamped and stirred so as to produce a homogeneous mass.

The expansion joints of this dam are spaced at intervals of only 12 m. The water face is drained by very porous cement piping placed horizontally.

In this latest example of a German concrete dam the outer surfaces have been dealt with so as to simplify the construction and reduce its cost, while simultaneously improving the quality of the concrete from the point of view of impermeability and resistance to atmospheric action.

The latest examples in Germany of concrete barrages are the Hohenwarte dam below the Saale Valley dam and the Lütsche-Valley dam near Oberhof in Thuringia both of which are under construction.

The *Hohenwarte Valley* dam is designed to be 75 m high above the base of the wall and its top length will be 450 m. The binding agent selected is a mixture having a 60 per cent. content of trass Portland cement 40/60 and 40 per cent Thurament, both together being 285 kg/m^3 of ready concrete. Granite with a maximum mesh of 100 mm, is used as aggregate. The concrete was of plastic nature and transported by cable cranes. The installation of cooling arrangement in the concrete is intended so that the heat set free during the setting process can be dispersed. The concreting work had not started at time of writing this report.

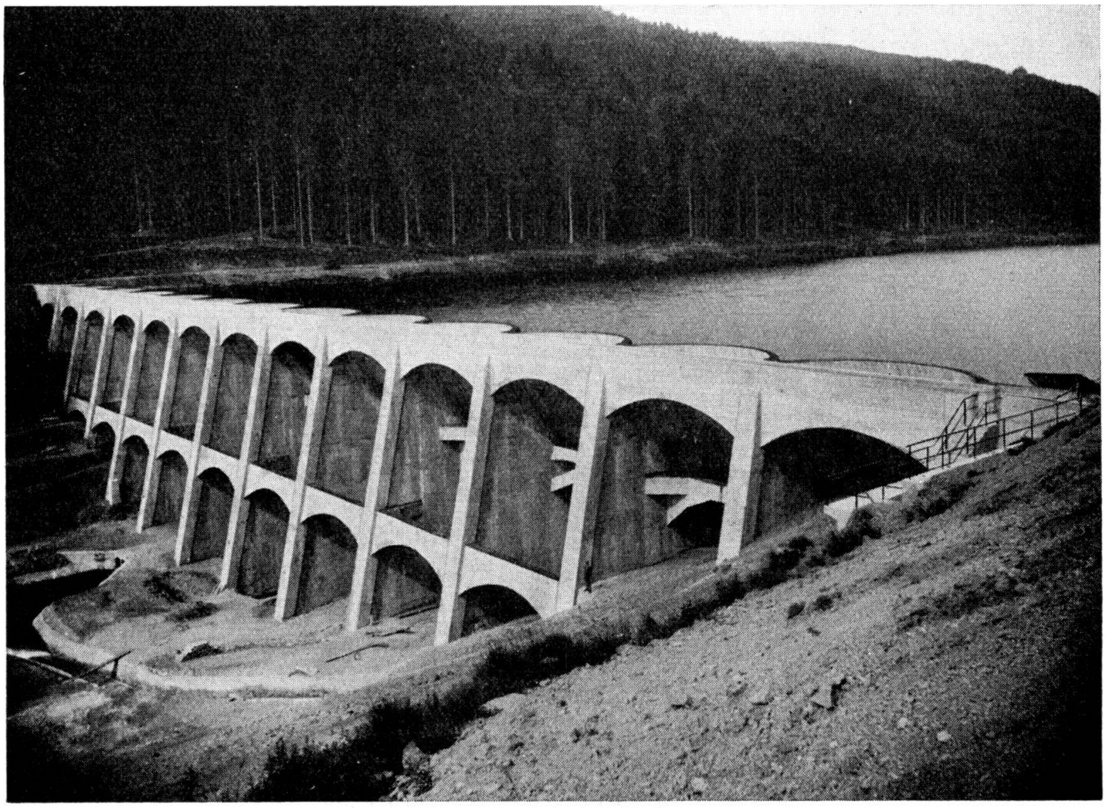

Fig. 3.
Linach barrage near Vöhrenbach

The outer faces of the wall will not be plastered or painted, as the concrete to be used will itself be sufficiently watertight. In order to remove the seepage a draining system will be provided behind the upstream face.

The *Lütsche Valley* dam is 35 m high from base of the wall to its crown, the latter is 210 m long. Portland cement alone will be used as binder when making the concrete and there will be no addition of a hydraulic binder such a trass or Thurament. The cement content for the core concrete will be 240 kg/m^3, while for the facing concrete on the water and downstream faces, which, will be 1 m thick, 300 kg/m^3 will be used. The aggregate used will be broken porphyry with a mesh of 70 mm. In this case, just as for the Hohenwarte Valley dam, the concrete will be transported in the form of plastic concrete with a cable crane and then made more compact by tamping and rodding.

Apart from the facing concrete referred to above, the outer surfaces will not be subjected to any other treatment to render them more watertight. The seepage will be collected by a system of drain pipes placed behind the water face.

The *Linach Valley* dam at *Vöhrenbach* (Baden Black Forest) (Fig. 3) ist the only dam in Germany built with buttness piers (with rows of arching) and in reinforced concrete. It is made of plastic tamped concrete of a composition in keeping with its closely spaced reinforcement. The upstream face of the arching has a layer of gunite rendering and several protective coats of paint.

The whole structure is watertight and does not appear to be defective in any way. If this style of construction has not been much in favour in Germany heretofore, no doubt it is due to the fact that it is an expensive method and building is complicated, so that little saving can be effected with it.

2. Earthen dams with concrete and concrete core walls.

Later on it was found that the rolled stone and earthwork barrages with watertight core walls, a few examples of which were first made in North America, enabled a saving in cost and this style was therefore developed and a number of these dams have been built quite recently.

Fig. 4.

Sorpe barrage, cross section (see Deutsche Wasserwirtschaft, march 1932, p. 42, fig. 1).

The *Sorpe Valley* dam (Fig. 4) in the basin of the Ruhr and the two Harz Valley dams at *Söse* and in the *Oder-Valley*, furthermore the *Kall Valley* dam (Fig. 5), not completed until 1935, are built according to the original design of this type of barrage. The slender core wall is subdivided by expansion joints into blocks of 24 (in the case of Sorpe) to 20 and 15 m. (Kall Valley dam.) It appeared that wider spacing was permissible than in the case of walls without earthwork, as the core walls are embedded in the soil.

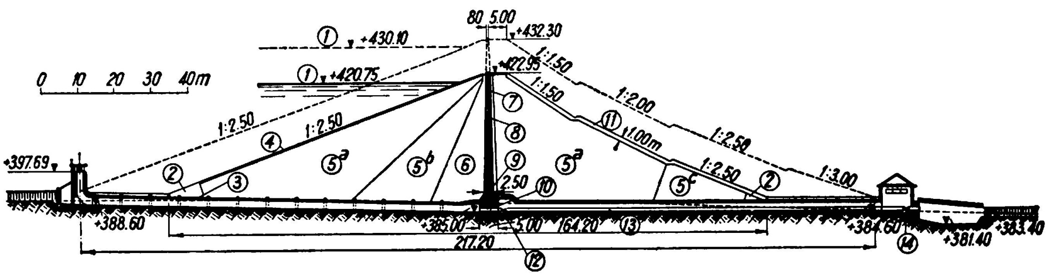

Fig. 5.

Kalltal barrage, cross section through scour outlet.

1) Flood water overflow.
2) Rubble stone packing.
3) Rock.
4) Stone pitching 0.60 m.
5a) Stone deposit coarse.
5b) Stone deposit fine.
5c) Stone deposit very coarse.
6) Deteriorated clay.
7) Metal.
8) Concrete core.
9) Sliding joint.
10) Inspection passage.
11) Turf-clad earth cover.
12) Throttle valve (closes automatically if pipe fails).
13) 2 Pipes 650 and 1100 mm diam.
14) Ring valve.

The core wall of the Sorpe Valley dam is made of the following ingredients calculated per m^3: 180 kg blast furnace cement, 60 kg artificial trass lime, 1890 kg aggregate having a grain of 0 to 60 mm with a water-cement ratio = 1,0. On the water face there is a 0.50 m thick layer of facing concrete consisting of 225 kg blast furnace cement, 75 kg trass lime and the same aggregate as above. For groating the joints between the blocks the concrete for this work received a richer mixture by adding 250 kg blast furnace cement and 75 kg trass per m^3. A facing of gunite, followed by several coats of paint, was applied to the facing concrete, while vertical drain pipes were built inside the core wall, passing out downstream at the projecting service passage at the foot of the wall. The Sorpe core wall ist excellent from the point of view of staunchness, however, it should be mentioned that so far the water has not reached its maximum height. (Full year's storage.)

The fundamental structural problem is that of giving passive earth pressure to the core wall from filling on the downstream face; this is necessary in order to strengthen the core wall so that it can resist the pressure of the remaining half of the dam and of the water pressure. The existence of this passive pressure of the earth rather leads one to assume that there is a certain displacements of the wall which, it would appear, cannot take place without a certain amount of overstressing of the material.

Attempts have been made to meet this by building channels in the neighbourhood of the foot of the wall (Söse, Oder and Kall Valley dams). These channels have been designed in the style of mere tipping or sliding grooves, or also as a combination of both. They are slightly curved and slightly inclined, and have an intermediate layer of coarse asphalt; in the case of the Kall Valley dam sliding plates have been added. So far, however, no definite opinion is available regarding the precise value of this method.[3]

Attempts are at present being made in Germany to develop core wall dams of this kind by substituting for the rigid concrete walls a more flexible steel wall and using concrete merely for the base of the wall where it joins bedrock; the concrete wall being only a few metres high.

In the *Ruhr Valley barrage* near *Schwammenauel*, the upstream coping consists of a steel wall made of horizontally arranged sheet piling; in the new *Bever Valley dam* (Rhineland) the steel wall is vertically arranged and made of corrugated sheet metal 8 mm thick welded together.

We have no information as yet as to how these dams, parts of which are still under construction meet the demand made on them.

II. *Experience and views regarding the main outstanding problems of dam construction in concrete, in Germany.*

Introductory Observation.

It is probable that decisions influenced by experience depend more on time and place in barrage construction than in any other branch of the civil engineering. Climatic, hydrological, geological and even morphological circumstances vary from one place to the next and have a marked influence on demands made on concrete,

[3] The Barrage Committee of the State Association of the German Water Supply is engaged on compiling a technical publication on the subject.

the possibilities of its composition, its manufacture and its transport. Important economic factors, such as wages, recruiting of skilled labour, cost of raw material may exert temporarily fluctuating influences. This preliminary remark refers equally to all that follows.

1. Preparation and manufacture of the concrete.

The *aggregate* had for the most part to be obtained for German barrage construction by crushing broken stone; the only difficulties encountered were that the stone quarries concerned were occasionaly inadequately equipped to meet the large demands or that the good strata had been exhausted.

The shape of the grain and its grading for size was always given the attention they deserved on account of their great influence on strength, compactness, density and weight as compared to volume of the concrete. In the past, it was the practice to use the curve for grain separation (Graf's curve) (Fig. 2), in which the largest grain is of such size as to pass through the largest mesh available, and as time went on the size of grain was increased. In the Bleiloch dam it is 60, Schluchsee and Zillierbach 80, while the specifications for the Saale Valley dam at Hohenwarte (at present under construction) laid down 100 mm.

Quite frequently rubble blocks ("plum" stones) were embedded in the plastic concrete. At Schwarzenbach the size of these was up to 2 m^3 (in practice the maximum is mostly 1.5 m^3, for large blocks the minimum size is 0.1 m^3) and the average volume ratio for these stones is 20 per cent. 30 is considered (according to Heintze)[4] the technical maximum, while the figure for the economic maximum is 25 per cent. This inclusion of stones (also introduced at Schluchsee and Schwarzenbach) reduces the need of a binder agent for the wall, increases staunchness, constancy of volume and specific weight of the concrete, and all these factors combined allow of the cross sectional dimensions being reduced, while the quality of the horizontal service channels is improved and the need for expensive dovetailing is eliminated. Plant requirements for the preparation of the aggregate and the manufacture of the concrete are also simplified, but special washing and transport apparatus become necessary. In terms of money, however, the method is advantageous. Transport of rubble stone also complicates the main building operations, considered as a whole, however, this method as applied at Schwarzenbach can be recommended. An indispensable condition in this connection is the compostion of the blocks used; their shape must be suitable and they must be sound and free from cracks. The barrages in North Germany have not as a rule complied the assumptions made.

The same principle as that underlying the use of large blocks is followed in another direction, in particular by the proposal[5] put forward by the "Austrian school", providing for the utilisation of intermittend mesh sizes with an unsteady and grading distribution. This idea was partially adopted for the first time in Germany when building the Zillierbach dam[6] (Fig. 2).

The brocken metal was graded into five sizes: 0—3, 3—7, 7—15, 15—40, 40—80 mm. After this the greater part of the 7—15 grain material was collected and

[4] Bautechnik, 26. XI. 1926.

[5] *O. Stern:* „Neue Grundlagen der Betonzusammensetzung" (New Principles for Concrete Composition), Z. Oe. I. Q. V. 1930, Issue 31/32.

[6] *Forner:* Bautechnik 29. V. 1936.

fed into a sand mill for still further desintrigation, so that a uniform sand 17 per cent resulted. The remaining requirement of sand was covered by adding ordinary pit sand brought to the site by rail. The various amounts of graded grain were put together just prior to charging the concrete mixer. The 80 mm mesh size found by preliminary experiment to be the maximum size permissible, because in the coarser mass there was a large amount of cracked pieces and these reduced the strength of the concrete.

The determining of the content of fine grain permissible called for the consideration of two opinions.

Up to the present there are two opinions as to the value fine sand as a filler. It has always been emphasised and its value was estimated so highly that where the amount of stone powder seemed inadequate, it was increased by further grinding and used as a "substitute for cement". Certain German engineers, however, had recognised long before the "impoverishing" effect of the stone powder, due to its extensive specific surface. This effect was recognised most clearly by the "Austrian school" and they studied the point thoroughly. Spindel (Vienna) has long since emphasised the injurious, loosening effect of all kinds of stone powder grain which is smaller than 5—7 times the grain diameter of the cement.[7] This application of the principle of grading the grain of the mixture: binder plus sand (d = 0,2—7 mm) has proved its value during laboratory tests for the manufacture of a dense concrete capable of withstanding inclement weather, and in particular, in connection with its application on a large scale in modern barrages built in the mountainous regions of Austria (Spullersee: 2 walls at over 1800 m level; put into commission in 1925, and Vermunt dam at over 1700 m level)[8].

Binding material. The standard requirements concerning selection and amount of the bindung material to be used in German barrage construction with its comparatively low height of retaining wall call mainly for constancy of volume, staunchness, weatherproof qualities and chemical resistance, since mechanical strength is always easier to obtain (the proportions are slightly different for barrage core-walls), and that is why attempts were made originally (following on the mortar composition for quarry retaining walls of rich lime and trass concrete advocated by Intze and his colleagues) to produce as cheap a concrete as possible while ensuring good binding qualities by introducing portland cement, lime and trass. However, when building the Schwarzenbach barrage, the idea of adding lime was dropped during execution, probably because it was found unsatisfactory and required longer time for setting. From then onwards lime ceased to be used as a concrete binding material for German barrage construction.[9] On the other hand, trass alone continued to be largely used in order to improve the cement or to produce a cheaper mixture (it is known to have no binding properties when used alone). Economic success depends to a very great extent on the facilities for transport to the districts where trass is found (Rhineland and Bavaria) and under the most fovourable con-

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[7] *Spindel:* Wasserdichter und beständiger Beton für Sperrmauern (Watertight and consistent concrete for retaining walls). B. & E. 5. X. 1932. Also Tonindustriezeitung 1913, No. 66, and Wasserwirtschaft, Vienna, 1933, No. 17/19 and 1935, No. 14/15.

[8] Cp. *B. Widmann*, Berlin: Deutsche Wasserwirtschaft, 1. VII. 1935.

[9] In the allied industry of Sluice Construction, however, it is used. Cp. *K. Ostendorf:* Die Bautechnik, 1927, Issue 39.

ditions such success is but moderate. With reference to improving the quality of concrete, laboratory experiments have proved that increased ductility and tensile strength of the mortar for heavy retaining walls are not decisive factors. The expansion of the concrete is increased by adding trass aggregate, the contraction is not reduced, indeed it is sometimes increased as a result of the increased requirement of water added to the cement-trass concrete. Opinions are divided concerning the diminishing of the setting heat by adding trass aggregate. It is generally conceded that trass has the valuable property of binding the excess lime in the cement and rendering the concrete more plastic so that it can be worked more easily.

There is no uniform practice in German barrage construction with regard to the addition of trass (even where the transport conditions are similar). When building the Bleiloch Valley dam an artificial substitute for trass — powder forming part of basic blast furnace slag — was used as »Thurament«.

The preliminary mixture of the binder which is necessary when using trass and similar material is being increasingly replaced by producing commercially trass-portland cement (Trapo) which has property of improving the concrete.

Where river water has injurious effects on the wall (very soft water) *blast furnace cement* was used on various occasions (Schluchsee, Schwarza) and the results so far achieved were found to be very satisfactory. As a matter of fact the seepage through the walls of the Schluchsee dam does contain a considerable amount of lime, however, as experiments have not yet been concluded, no final verdict can be expressed.

Alumina cement was used for patching and facing a small retaining wall in the southern parts of the Black Forest; this wall was built of portland cement and had been damaged by the soft water attacking it for a period of four years. Before long, however, signs of expansion appeared (continous scaling). Meanwhile certain scientific investigations have confirmed that the volume of alumina cement does not remain constant in water.

Recent requirements concerning fineness of grain in binding material have led to the tightening up of regulations so that the 900-mesh sieve has now been eliminated in favour of the 3600-mesh. It is pretty certain that this improvement of the binder with an appropriate limiting of the finest grain in the aggregate would lead to the manufacture of a more compact and weatherproof mortar and so produce better concreted surfaces.

Admixture of water. If too much water is added this has the bad effect of reducing crack resistance and density of the cast concrete and this fact was soon recognised by the German barrage builders.

The general practice since the construction of the Schluchsee and Schwarza dams, and including these has been to use only concrete of "soft" (plastic) composition; in this connection, however, there are still divergencies of opinion when gauging what in practice is the best content of water according to the wide margin offered by the alternative possibilities of using cast concrete or moist tamped concrete. When tenders were recently invited for a project the specification laid down a soft concrete with a flow in the channels of at least 27° incline (1:2), however, one of the groups of German engineers holds the view that the water content should be reduced as far as is compatible with the other building conditions and also that tamping should be carried out in moderation. These engineers applied these latter principles when building the Vermunt dam in the Austrian mountains, and they were very successful in their application of concrete con struction with mixtures of "very moist" core having a tensile strength of 150 kg/m³ and facing concrete which was "sufficiently plastic for tamping purposes" and had a tensile strength of 300 kg/m³. In spite of the various opinions at present extant, there is agreement that the stiffness of the concrete should no longer be governed by the design of the mixer, but that the mixer should depend on the various building requirements prescribing the most suitable degree of stiffness.

Mixing and pouring. For some time past the mixing process has been automatic for dam construction in Germany, and if necessary adjustable, weighing and measuring appliances being specified for binding, aggregate and water.[10] Continously working mixers are frequently prohibited. The time for mixing is restricted as far as possible. When constructing the Zillierbach dam the period prescribed was only

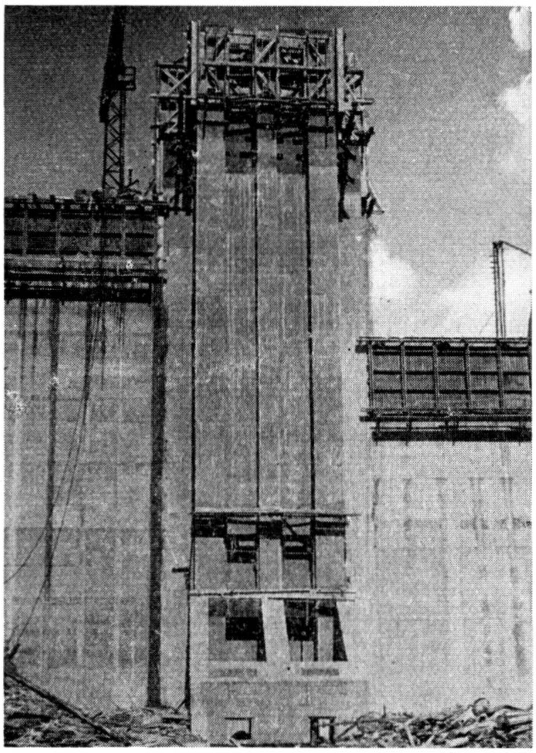

Fig. 6.
Zillierbach barrage.
Downstream face.

¾ to one minute (total time of operation 3 minutes); the concrete produced was excellent from every point of view (Fig. 6, Zillierbach dam, seen from downstream). Nowadays the mixing plant and the dam being erected are connected by cable railways (Schwarzenbach dam and again recently the dams being erected at Hohenwarte and Lütsche) or by trolleys and conveyors, which are constructed on piers or run along the future top of the wall (Bleiloch[11] Zillierbach), or else by adjustable rotating lifts placed on the downstream face at the foot of the dam, in connection with which short channels or conveying belts are used (Vermunt[12]). At Zillierbach rotary tower cranes are employed. It is impossible to go fully into the very varied

[10] *O. Graf:* Bautechnik, 10. V. 1929.

[11] *W. Kesselheim:* Bauingenieur, 1932, Issue 13/16.

[12] *Habild:* Z. V. D. I., 20. VI. 1931, and *Widmann:* Deutsche Wasserwirtschaft, 1935, Issue 7.

and comprehensive means offered in this respect, however, we would just mention as particularly interesting the use of vertically articulated pipes on the service bridge of the Bleiloch Valley dam which is situated at a good height. In this case, where there was a free fall of 40 m, the danger of the concrete demixing was absolutely prevented by placing baffles on the inside, these acting as gravity mixers. The pipes fed distribution appliances of two to three branches and fixed to the working plattforms (Fig. 7).

So far the *concrete pump* has not been used in dam construction in Germany, but it has been successfully applied in connection with other concrete constructions in which a maximum mesh up to 80 mm was used[13]. The amount of water added had to be such that the concrete poured out the end of the main pipe in a "plastic" state. Horizontal conveying distance of 250 m was easily attained.

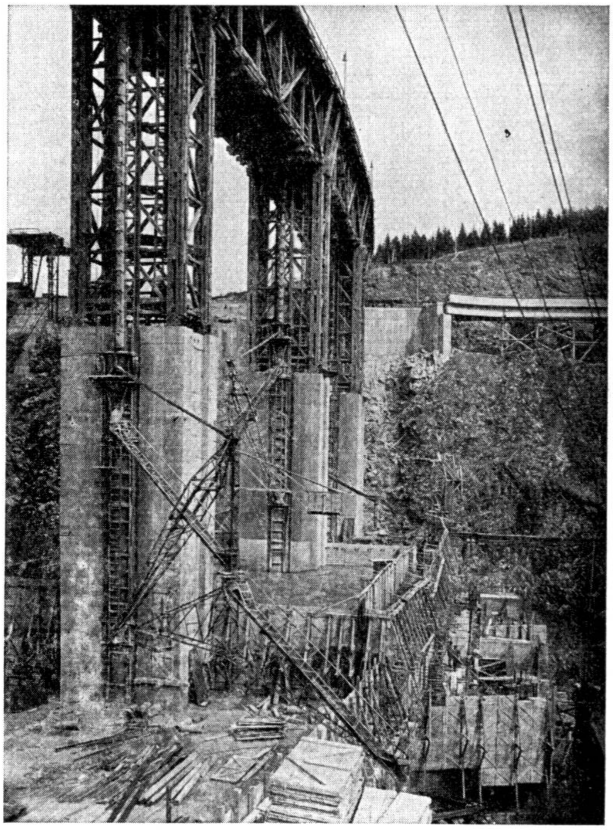

Fig. 7.
Saaletal barrage. Concrete pouring plant.

The output of work was considerably increased with these modern appliances and material. The monthly output at the Bleiloch dam, for instance (210,000 m^3 of concrete) was (incl. erection of the power house) 29,600 m^3; the daily peak figure was 1540 m^3 (two shifts working nine hours). In the case of the Vermunt dam (145,000 m^3 of tamped concrete) the monthly output reached the figure of 28,600 m^3 with a daily peak of 1960 m^3 (20 working hours).

Block arrangement, dovetailing and height of cast blocks are treated in Germany much the same as in other countries. It is interesting to note that experience has shown that absolutely rectangular groundplan dovetailing for the expansion joints

[13] *W. Kesselheim:* Bauingenieur, 1932, Issue 13/16.

occasionally leads to oblique tearing (shear) at the corners. (This was noted on the downstream surface where these perpendicular corners project.) Trapeze shaped form and other dovetailing has frequently been recommended and we should like to support that view.

The distances between the permanent *expansion joints* have been reduced from 30 and 25 m (Agger, Schwarzenbach, Bleiloch) to lower figures (Schluchsee 15,5 to 12; Schwarza 20 to 10; Zillierbach 12 m). Definite cracks were observed in the Agger and the Bleiloch dams. On account of the facing on both sides of the Schwarzenbach dam, it ist not possible to state definitely whether cracks have formed or whether the beneficial effects of the insertion of plums (20 per cent reduction of concrete) has prevented their formation.

At Schluchsee it was found that expansion joints which did not extend as far as the foot of the masonry subsequently reached that point by cracking. It is interesting to note that the cracks in the Agger Valley dam (Fig. 8) started at the

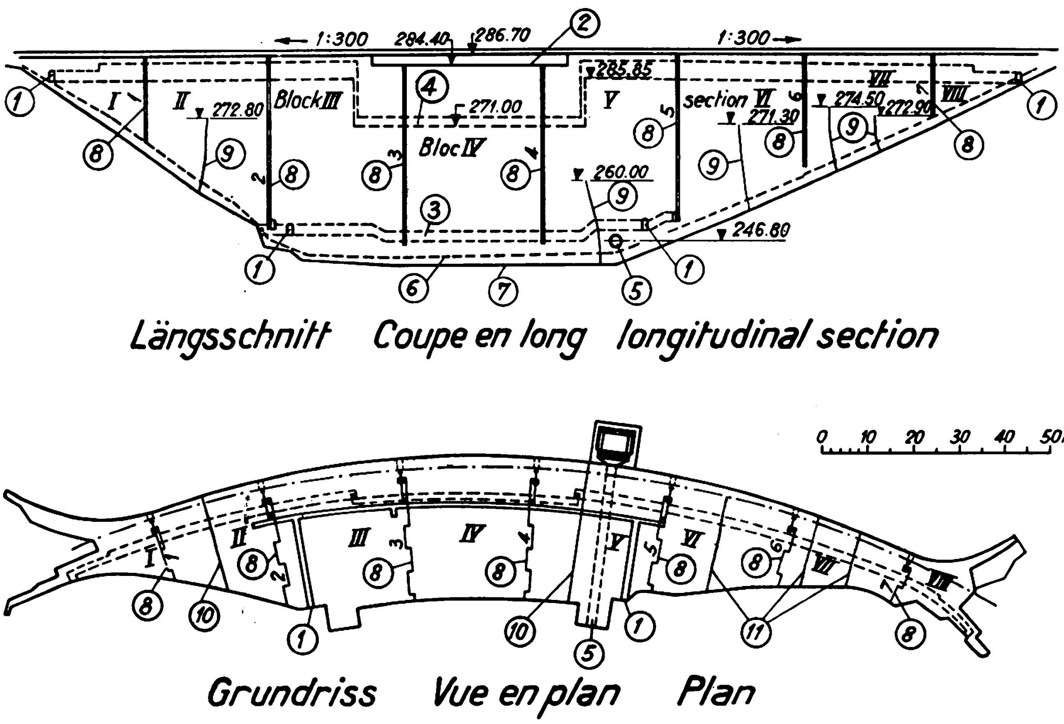

Fig. 8.

Aggertal barrage, cracks and joints.

1) Entrance to gallery.
2) Overflow.
3) Inspection gallery.
4) Upper Inspection gallery.
5) Outlet.
6) Rock surface.
7) Rock bottom.
8) Joint.
9) Crack.
10) Observed line of crack.
11) Probable line of crack.

base but did not reach the top. This phenomenon is not in keeping with the theory of crack formation in straight walls and is probably caused by the curved ground plan arrangement of the Agger dam and the increased heating and slower liberation of heat of the heavy lower part of the wall. Another point worth mentioning, although it is not easy to explain it, is that cracks seem to start mostly in hollow parts of the structure and in passages which are at right angles to the axis of the wall (Dreiläger Valley dam). In this connection we would mention that several of the modern German rubble-stone walls have cracks and other defects producing leakage[14]). These walls, with one exception, are not provided with any special protective means (joints) beyond the curved ground plan.

Lack of space prevents our dealing fully with the problem of making the expansion joints leakproof. As a matter of fact this is done much along the same lines as in other countries.

Sheet copper and moulded asphalt, commonly used to make joints staunch, was sometimes inserted in the shape of a lyre and sometimes in Z-shape (Schluchsee). A second packing device was often added in the shape of a vertical reinforced concrete tie of a dovetail section. Difficulties often arose when inserting these copper plates, with the result that the surrounding "high grade" concrete turned out porous. The designer of the Zillierbach dam accordingly, and on the strength of his experiences in connection with that dam, suggested facilitating the insertion of the packing and the improvement of its density by adding a grooved piece (sheet metal and angle pieces) in addition to the copper plate.[15] This device appears to offer a satisfactory solution (Fig. 9). A detailed report on Expansion Joints was drawn up by Link for the World Power Conference, Washington, 1936.

Fig. 9.

Caulking joint plate and built-in box joint.
(see „Deutsche Wasserwirtschaft, Aug. 1936. Paper of Mr. Forner, Fig. 2).

Facing. It was found that the steel formwork in the Zillierbach dam had been calculated too weak because building operation progressed more quickly than had been anticipated and the accumulation of blocks proved too heavy; and also because the blast furnace cement and soft concrete, which set very slowly, set up greater lateral pressure than had been anticipated. In this dam the steel formwork was applied only to the outer faces where clean surfaces and great staunchness were imperative (Fig. 10), and this proved very successful. The surfaces of the blocks on the inside of the wall were lined with a wooden formwork (patented by the contractors) having frame dimensions of 3.3 by 3.0 m with cross beams. These proved very satisfactory, for they could quickly and easily be replaced. It was found necessary to arrange and supervise very carefully the stiffening or anchorage of the formwork, so as to get surfaces that were absolutely true to plan.

Subsequent treatment, including moist coverings, spraying, etc. are nowadays considered as very important in view of what experience has taught us. In the specifications for the new Saale Valley dam at Hohenwarte, for instance, constant watering of the faced blocks for a period of two to three months is insisted on. Arrangements for lowering the setting temperatures of the large blocks have not been applied up to the present; they will, however, in the construction of the Hohenwarte dam. Moreover, it is here projected to instal a temperature-regulating device in the concrete-mixing works, so that the concrete will be delivered throughout the building period at a constant temperature.

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[14] Cp. *Ludin:* Bericht über die Außenflächen der deutschen Staumauern, Report on the outer faces of German retaining walls, World Power Conference, Washington 1936, Congress on Barrages.

[15] Deutsche Wasserwirtschaft, August 1936.

2. Surface treatment and surface facing, Protection, staunching and drainage of the concrete.

Facing of the downstream surface with natural stone has not been done excepting in the case of the Schwarzenbach, Agger Valley and Kleine Brändbach dams. Frost and weather do not seem to have had any markedly bad effects on the earlier concrete retaining walls with unfaced downstream surfaces (Schluchsee, Bleiloch). The same may be said of the upstream face; the only one that has a protective covering is the Schwarzenbach dam.

The temperature of the air varies between + 37 and — 28° C in the case of the Saale Valley dam. Thus the climatic conditions to which most of the German dams are subjected are almost as severe as those to which Alpine barrages are exposed.

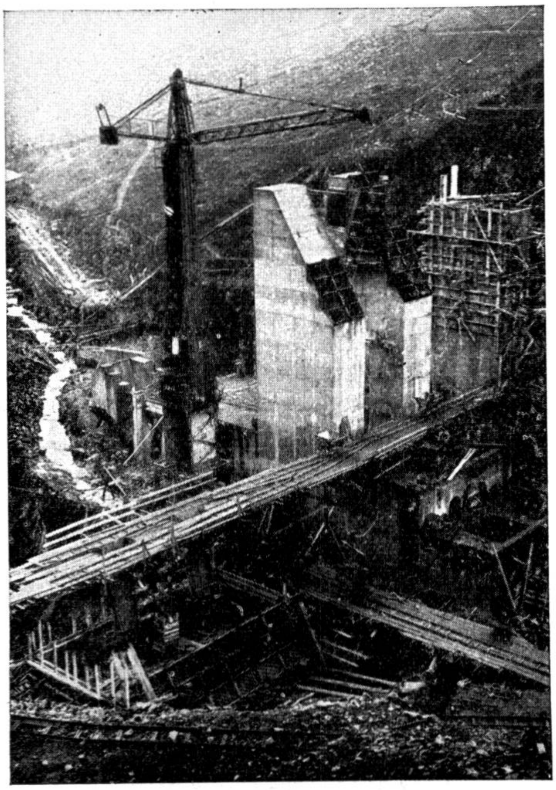

Fig. 10.
Zillierbach barrage.
Erection of steelforms.

No final decision has been reached in Germany regarding the advantage or disadvantage of using uniformly compact concrete for the whole thickness of the wall or porous coarse concrete with dense facing concrete on either face. Both the practical (list 1) and the theoretical aspect have still to be decided, however, opinion seems to be gaining ground in favour of facing concrete. Experience gained at the Bleiloch dam point to the advisability for technical reasons (crack formation) of not making the whole wall of a mixture having a high content of binder and water. (The Kriebstein retaining wall cannot be quoted as a satisfactory proof on account of its limited dimensions, while the Schwarzenbach wall is equally unsatisfactory because of its peculiar design (protective facing), based ou historical considerations.)

Facing concrete with a core having a medium to low content of binding material is more advantageous from the economic point of view.

If previous experience, in particular that of the "Austrian school", be followed, facing concrete which is practically watertight and able to withstand bad weather conditions can certainly be manufactured. Opinion is divided as to whether a further coating (gunite) should be given to the concrete. The Zillierbach Valley dam has been made without any such facing on the downstream wall, which faces a north aspect, and will serve therefore as a good test. The facing concrete of the Schluchsee and Schwarza dams were faced with gunite concrete to make them more watertight, but seepage has nevertheless occurred in small quantities, it is true, but this will have to be closely watched because the water contains dissolved lime. It is believed that the water leaks chiefly through the expansion joints. Investigation is still proceeding.

The two to three *protective coats* applied in the past to plastering or to facing concrete, mainly to resist the effects of storage water and to fill up interstices have mostly proved unsatisfatory as regards withstanding bad weather.

Recently various products have been placed on the market which appear to answer the purpose better. For instance, there is a (patented) plaster made of asphalt emulsion, asbestos fibre and sand which has been used for the rubble-stone wall at Neunzehnhain II, near Chemnitz in Saxony; this has been applied in a 2 mm thick layer. Other experiments have also been made lately in other branches of the concrete industry, e. g. with a concrete called "Bitukret" and these experiments deserve to be carefully watched in the interest of barrage construction. This material (patented) which is loosely applied is a mixture of cement, sand and diluted bituminous emulsion called "Tunol", which is stated to offer the following advantages: absolute resistance to saline agents, acids, etc., good adhesion to concrete or to walls and brickwork, in particular when the basic material is moist, and further, resistance to frost, heat and mechanical stresses.

Although the concrete surfaces are protected above all from the effects of water by the compactness of the concrete itself, it would seem very important, particularly where water exerts a specific wearing action, to apply some superficial coating which can resist chemical action. Bituminous products of the kind mentioned above may have the qualities desired. So far it has not been considered necessary in Germany to take measures which are as drastic and as costly as *steel plate facing*.

The facing concrete on the water face was generally applied in a uniform thickness, but thicker than on the downstream face (for instance, cement: 275 and 300 kg/m^3 and 0.75 to 1.00 m on the downstream face and 1.2 to 1.5 m on the upstream face) at Schluchsee, Schwarza and Zillierbach.

The facing concrete was applied at the same time as the core concrete without any intermediate formwork, unless perhaps drawn sheet metal, so that irregular dovetailing and a perfect grip was ensured for both kinds of concrete. No difficulties arose in this connection.

Draining. Besides the draining of the base of the wall, which is so important for static reasons, most German retaining walls are also provided with a drainage system on the pressure face. This is effected by means of a vertical system of collecting pipes (drains) generally placed at intervals of 1.5 to $\sim$ 4.0 and situated behind the water face of the wall. The seepage caught in this way is led to the service passages, which latter are general nowadays, and from there the water escapes into the open. The arrangement of the collecting pipes is suited to the concrete structure, mostly as near the horizontal as possible (instead of vertically as used to be the practice) and they often meet in the shafts of the expansion joints (see below). Instead of ordinary earthen or cement pipes (whose joints are left unpacked) the custom nowadays is to use blocks of coarse pored concrete hollowed out to resemble pipes of 10 cm diameter, or similar concrete pipes, which are protected by a covering of newspaper before the cement batch is poured over them when concreting takes place. These pipes are spaced at intervals of 1.5 and ∾ 5 m.

No internal drainage was provided for the Agger Valley dam, and this would appear justified when using porous core concrete covered with thoroughly compact facing concrete, as the core concrete can do the work of the collecting pipes provided it is properly drained by a few collecting pipes in a nearly horizontal position or provided the service passages are so drained.

The *drainage* of the *top of the wall* and the process of making it *watertight* should receive closer attention in view of some unsatisfactory experiences. Reports have been received that on the downstream face of the Agger dam water trickled down behind the stone facing during the day and that the torcrete plastering on both downstream and water faces of the Schwarzenbach dam suffered from moisture with the result that damaged was later caused by frost.

It was not necessary to supplement the *watertight linings* in any of the dams except at Schluchsee and Schwarza, where further cement was poured into the interstices of the expansion joints, but not with entire success.

3. Attendance and inspection.

In order to be able to watch the heat distribution and changes of volume on the inside of the wall at the Bleiloch dam, 39 telemeters were built in, and subsequently 30 at the Schluchsee dam. A provisional report has been drawn up giving the measurements recorded[16]; these are not very different from the results obtained with dams in other countries.

The *movement* of certain parts of the walls, in particular the tops, have been systematically observed in Germany (since the time of *Intze*) by means of simple apparatus. Lately trigonometrical precision measurements have also been made.[17] The concession archives of Baden contain repeated mention of surveys carried out on these lines and the results are always entered in the barrage records. (This procedure has now been introduced all over Germany.)

Arrangements for observing the pressure of the seepage and of the leakage were made a long time ago and this observation is being carefully carried out. It is limited to the base of the wall, as experience has shown that in the case of rubble-stone walls there is no apparent water pressure provided the usual measures for preventing leakage and for draining the walls are applied. On the other hand, basic water pressure is nearly always to be found at a height dependent on the height of the reservoir. In no case did this lead to any unusual phenomena. Circumstantial reports have been drawn up concerning the retaining walls of the Oester, the Eder and the Möhne.

The design and construction of dams are subject to strict State control and the regulations governing design, construction and operation of dams were laid down in a revised form a few years ago. These instructions have proved exceedingly satisfactory.

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[16] *Probst:* Deutsche Wasserwirtschaft, 1932, Part 7/8.

[17] Report by *Walther* in Bauingenieur, 5. III. 1927.

Table 1a. Concrete Barrages in Germany.

No.	Built in	Place, Water River	Height of dam-head above			Length in	Radius in	Up-stream face looks	Thickness of dam		Slope of downst. / upst. face vert.: horiz. = 1:	
			Sea-level m	Found-ation m	River-bed m	m	m		on top	on river-bed	downst.	upst.
1	1909—1911	Aachen *Dreiläger*	393.00	89.5	32	300	350	W.	3.00	29.00	0.591	0.10
2	1922–1926	Forbach *Schwarzenbach*	670 00	67	50	400	400	S. E.	6.00	39.00	0.711	0.031
3	1927—1928	Oberberg *Agger*	286.50	45	43	225	225	S.	6.50	29.00	0.647	0.05
4	1927—1929	Kriebstein *Zschopau* (Saxony)	214.25	30	23	230	225	N. N. E.	4.00	17.30	0.848	0.04
5	1929—1932	Seebrugg *Schluchsee* and Schwarza	931.50	45	35	240	straight with break on centre	W.S.W.	3.70	27.00	0.72	0.03
6	1928–1931	Schwarzabruch *Schwarza*	724.75	43	33	158	140	S. W.	3.70	25.50	0.72	0.03
7	1926—1932	Saalburg *Saale* on kl. Bleiloch	412.00	70	60	215	300	N. N. E.	6.70	46.15	0.69	0.02
8	1934—1935	Wernigerode *Zillierbach*	473.00	47	39	172.5	∞	N.	2.00	26.52	0.63	0.05
9	1935—	Gräfenrode *Lütsche*	583.50	35	25.10	210			3.80	16.75	0.71	
10	1935—	Hohenwarte *Saale*	306.40	75	67.20	450	400	W.	7.20	49.00	0.71	0 02

Table 1b. Concrete Barrages in Germany.

No.	Expansion joints: Extent m	Expansion joints: Interval m	Facing of upstream surface: Plastering: thickness in mm	Facing of upstream surface: Protective coating mm	Facing of upstream surface: Protective covering	Facing of upstream surface: Concrete facing: Thickness mm	Facing of upstream surface: Concrete facing: Cement kg/m³	Drainage of dam	Facing of downstream surface: Natural stone	Facing of downstream surface: Concrete facing: Thickness m	Facing of downstream surface: Concrete facing: Cement kg/m³
1	unknown	1 vertical joint left	unknown	affired with Siderosthen	yes	—	—	yes	yes	—	—
2	mid-dam 57,00 m high	centre block 36 m adj. 27 and 25 m	Gunite 25	from 606 — 630 three-fold cardboard layers under covering	yes	—	—	yes horizontal	yes	—	—
3	from top to 3 m above base	30.00	—	—	—	2 0–2.50	275 kg H G. C. + 83 kg Tr.	—	yes	—	—
4	from top to 1,5 m above base	20—25	Gunite with smoothened surface	Inertol 3 applications	—	—	—	yes	—	—	—
5	from top to base	12—15.50	Gunite 25	Inertol 2 applications	—	1.50	275 H. G. C.	yes	—	0.75	275 H. G. C.
6	from top to base	10—20	Gunite 25	Inertol 2 applications	—	1.50	300	yes	—	1.00 planed	300 wood casing
7	from top to base	25	—	—	—	—	—	yes	—	—	—
8	from top to base	12	—	2 coats up to within 8 m of top	—	1.20	300	yes	—	1.00	300
9	from top to base	12—13	—	—	—	1.00	300	yes	—	1.00	300
10	from top to base	15	—	—	—	—	—	yes	—	—	—

Table 1c. Concrete Barrages in Germany,

No.	Experience under Service Conditions							Supplementary and Repair Work done
	Upstream face		Downstream face					
	Cracks	Damage by frost	Cracks	Damp patches	Leakage	Incrust-ation	Damage by frost	
1	2 vertical	—	2 vertical	few	from work joints	yes	Top facing of dam	New surfacing of dam-top with chips and bituminous emulsion
2	—	in Gunite plastering	—	—	—	—	—	Gunite plastering repaired below coping; otherwise nothing noteworthy
3	1—2 hair cracks in each block	slight	1—2 hair cracks in each block	in winter a few on top of dam	yes	—	—	—
4	—	Flaking	—	—	—	—	Flaking	
5	—	in Gunite	—	Dampness in all joints	yes	—	—	Cement injections
6	—	—	—	—	yes	yes	—	Cement injections
7	4 large	—	4 large	—	yes	—	—	4 shrinkage cracks caulked
8	—	No reports available beg. 1936						
9	—	still under construction						
10	—	still under construction						

Table 2. Principal Materials used in German Concrete Barrage Construction.

No	Name of Barrage	Quantity of Concrete in m³	Type of Concrete: G. B. ¹)	Pl. or -W.B. ²)	St. B ³)	Binding: Cement kg/m³	H.-G. C. kg/m³	Aggregates kg/m³	Aggregates: Type of Stone	Granulation in mm	Mixtures: kg/m³ or parts	Admixture of water l/m³	Specific weight: γ_B t/m³
1	Dreiläger . . .	72 000	—	—	yes		—	Trass and lime	Quarzite		1/2 C. 2 1/2 Tr. 1 1/2 L. 7 Sd. 9 M.		—
2	Schwarzenbach .	297 000 with 20% "plum" stones	yes	—	—		—	Trass and lime			C. L. Tr. Sd. M. 1.0 0 5 1.0 4 6 units 1.1 0.4 0.8 4 6 " 1.0 0 0.6 5 7.5 " 1.0 0 0.44 4.6 6 9 "	250	2.25
3	Agger	100 000	—	yes	—	—	200 175	60 Trass 45 "	Graywacke	0—80	Aggregate 1850	212	2.33
4	Zschopau . . . near Kriebstein	82 000	yes	—	—	frist 200 then 180	—	Trass 75 65		0—60	Aggregate 1680	330	2.3
5	Schluchsee . . .	124 000	—	yes	—	—	up to 22 m below surface 200 above 22 175	—	Granite		Aggregate 2040	160—180	2.4
6	Schwarza . . .	52 000	—	yes	—	220	—	—	Granite		Aggregate 1920	160—180	2.31
7	Saale . . . . . on kl. Bleiloch	210 000	yes	—	—	b. 118 m. 105 t. 87	—	Thurament b 229 m. 205 t. 169		0—60	0—7 7—30 30—60 mm P.C. Thur. aggregates b 118 229 804 689 459 m. 105 205 859 687 459 t. 87 169 940 678 453	236—243	2.45
8	Zillierbach . . .	58 500	—	yes	—	200	—	—	Diabas and Porphyr	0—80		Wat - Cemt ratio 160—170	2.4
9	Lütsche	38 000	—	yes	—	Core 240 Facing (300)	—	—		0—70			
10	Hohenwarte . .	450 000	—	yes	—	$\underbrace{\text{Trapo} \quad\quad \text{Thurament}}_{\text{total 285}}$			Granite	0—100	(0.6 Trapo + 0.4 Thur.) : 2.48 Sd. 0—7 mm : 1.21 chips, 7—30 mm : 1.54 metal, 30—60 mm : 14.8 metal 60—100 mm	185—190	

¹) Wet liquid concrete. ²) Plastic Concrete. ³) Rammed moist.

Reference Works:

Schwarzenbach- and Schluchsee-Dams.

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Summary.

Modern dams made of concrete were first built in Germany in 1922. Originally cast concrete was used, but this was soon dropped in favour of soft concrete (plastic concrete). Meanwhile a dam built by German engineers in the Austrian Alps and made of tamped plastic concrete has found many advocates in Germany.

With regard to making the dams watertight, two schools of thought exist. One favours the construction of walls made of a uniformly compact mixture of concrete (with amounts of binding aggregate varying solely for two to three zones of height). The other opinion recommends making the wall core of concrete which is not absolutely impervious and of facing the water and downstream sides with additional plastering about 0,75 to $\sim$ 1.5 thick and consisting of high grade compact concrete, with an additional high grade coat, generally applied by spraying the downstream face.

Facings of natural stone were used only for the first three concrete dams; recently this practice has been dropped without any disadvantage to the barrages.

The retaining wall is mostly fitted with an arrangement of drainage pipes inserted in the wall.

A proper selection of size of grain for the aggregate of concrete is considered very important in order to obtain the right consistency of the concrete. The trend at present is to develop as far as possible the principle of intermittend (grading), and this is being done in various directions.

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VI 6

Reinforcement of Pressure Pipes for the Marèges Hydro-Electric Plant.

Umschnürung der Druckleitungen des Kraftwerkes in Marèges.

Le frettage des conduites forcées de l'usine hydro-électrique de Maréges.

M. Mary,
Ingénieur des Ponts et Chaussées, Paris.

On October 5th, 1935 the Minister of Public Works inaugurated the dam and hydro-electric plant at Marèges which have been built in connection with the electrification of the Paris-Orleans railway, under the charge of the Special Conservancy Service of Haute-Dordogne directed by Mr. Coyne, Ingénieur en Chef des Ponts et Chaussées.

This undertaking has been the subject of many articles in the technical press,[1] and its essential features will here briefly be recapitulated:

An arch dam 90 m high with a crest development of 247 m has been built across the Dordogne, 18 km below Bort-les-Orgues, forming an artificial lake of 230 hectares which contains 47 million m^3 of water, whereof 35 million m^3 can be utilised. Some 250 m below the dam the hydro-electric station has been built in an opening of the valley due to secondary streams; this contains four vertical sets of 34,000 KW each and two auxiliary sets of 2,300 KW.

The object of this paper is to describe in detail an entirely new method of construction which has been applied to part of the pressure pipe lines.

I. Statement of the problem.

The water is conveyed from the dam to the plant through undergrund tunnels. Intakes are arranged with their sills 35 m below the normal storage level, and two tunnels of 6.2 m internal diameter and 135 m length, on a slight gradient, lead from these. Each tunnel forks, at its lower end, into two pressure pipe lines, also underground; these are of 4.40 m internal diameter and of lengths varying between 120 and 150 m. These buried pipe lines terminate inside the station where they are connected to the turbines through sections of metal pipe 20 m long. The general arrangement of the plant is shown in Fig. 1 and 2.

The inflow tunnels carry a maximum internal pressure head equal to 45 m of water. They pass through a compacted mass of rock and it has not been considered necessary to reinforce the lining, which has an average total thickness

[1] See in particular Le Génie Civil of 7th July, 1934 and 26th October, 1935.

of 0.35 m of solid concrete produced by the cement gun (Fig. 3). The bond between the concrete and the rock, and the filling of fissures, has been made good by injection. From below the foot of the surge chambers the pressure becomes greater on account of the gradient of the tunnels and of the hammer blow due to sudden closure of the turbines: at the lower end it may amount to a head of 102.5 m, of which 72.5 m represents the static head and 30 m the effect of the hammer blow. Here, moreover, the rock is not so good, and

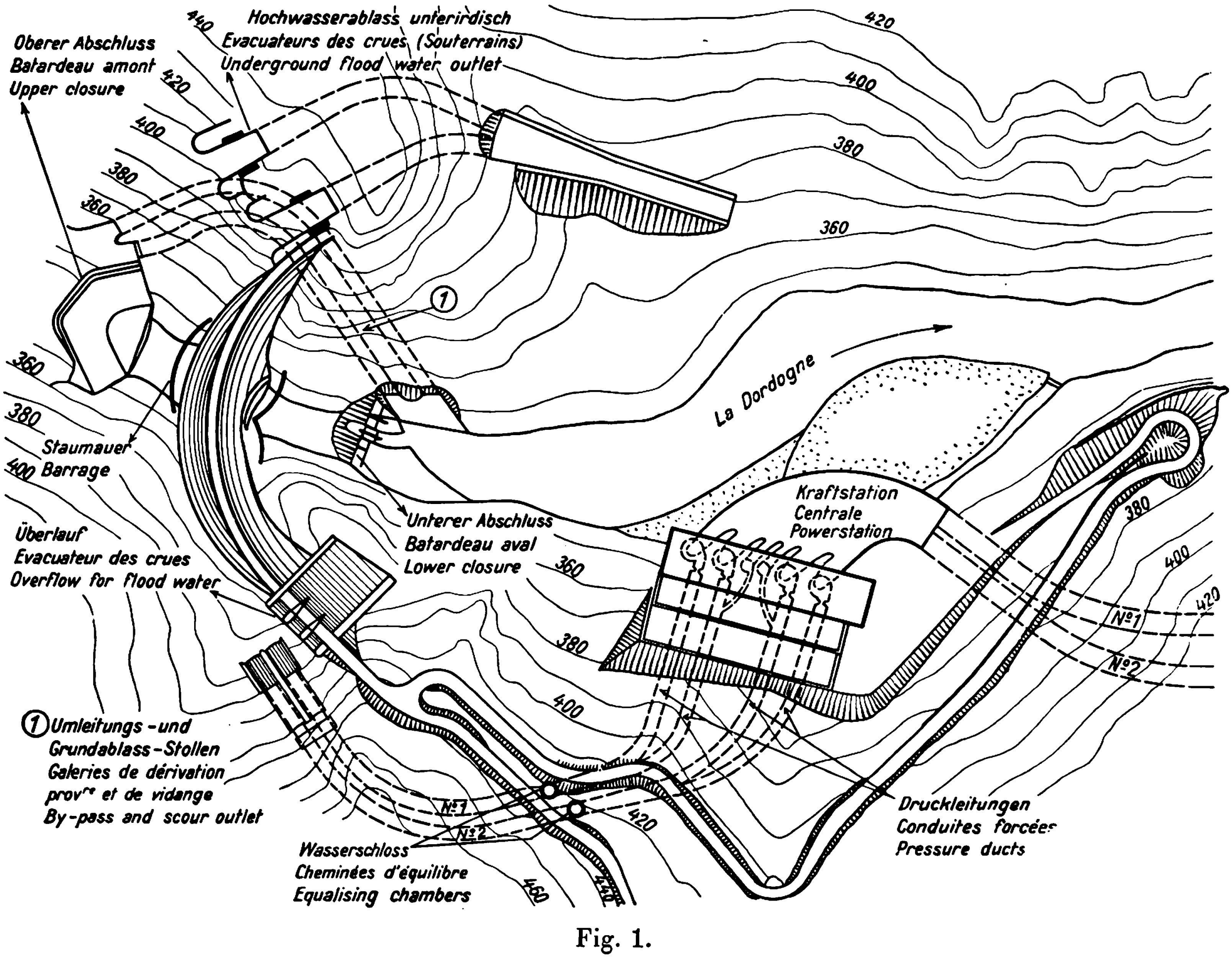


Fig. 1.

Barrage. Situation.

the lining has in consequence been strongly reinforced, consisting of 0.35 m thickness of ordinary block concrete in addition to reinforced gunite Fig. 4).

As the thickness of rock cover was considered great enough to allow of it being taken into account for the purpose of strength, the cross section of the steel hoops forming the reinforcement of the gunite has been so calculated that if the steel be assumed to take the whole of the internal pressure the stress in it will approximate to the elastic limit. Actually the stress is much lower on account of the increased strength of the structure due to the concrete lining and the rock cover, and while it is difficult to calculate the proportion of this relief it is certain that the stress in the steel could in no circumstances exceed the elastic limit. Here again perfect bonding of the concrete to the rock has been made certain by careful injection. Acoustic detectors, on the system invented by Mr. Coyne, have been installed at several points to allow the stress in the reinforcements to be checked during operation. On January 15th, 1936, this stress was about 2.5 kg/mm² and the pressure was such that if the whole of its effect had to be resisted by the steel alone the stress therein would have amounted to 10 kg/mm².

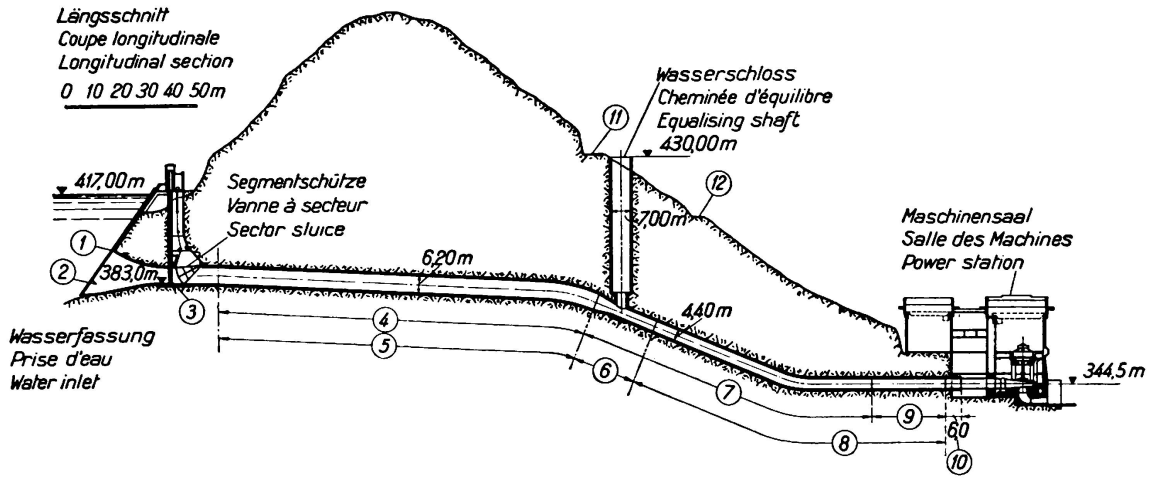


Fig. 2a.

Hydro-Electric plant of Marèges Water inlet and pressure duct N° 1.

1) Rake rails for cleaning grate.
2) Grate.
3) Emergency sluice.
4) Gunite lining.
5) Main duct.
6) Branching of ducts 1 and 2.
7) Reinforced gunite lining.
8) Duct N° 1.
9) Laterally reinforced portion.
10) Anchorage of metal duct.
11) Road to barrage.
12) Road to power station.

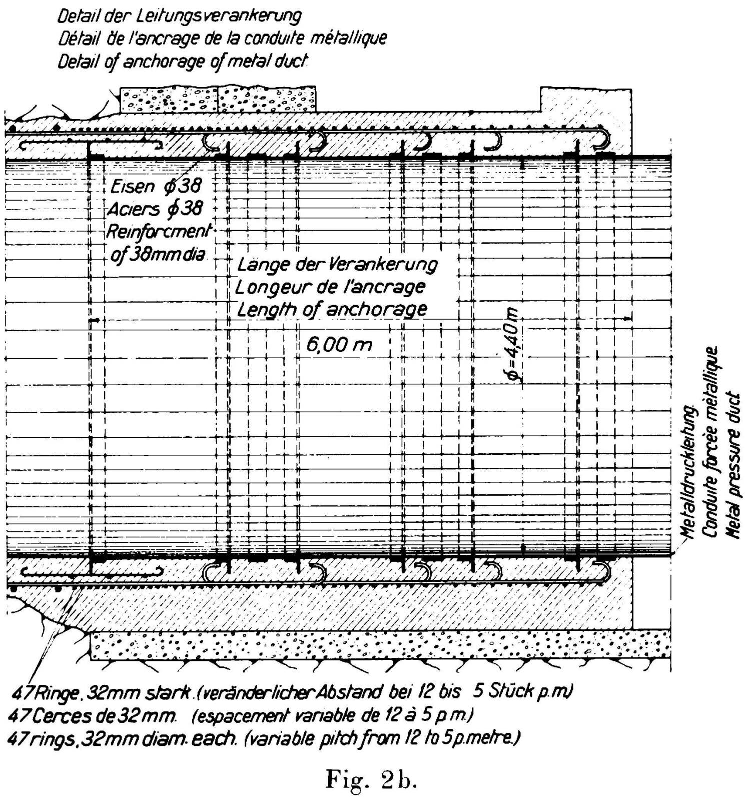


Fig. 2b.

Detail of anchorage of metal duct.

In the last 30 m run of the underground tunnels before discharging into the steel pipes they pass through rock of poor quality and the thickness of cover falls to 10 m. It is in this zone that the water pressure is at its greatest and may reach 102.5 m head including the hammer blow; the form of construction adopted over the remainder of the run did not, therefore, appear to afford a sufficient margin of safety here, and if for safety the resistance of the rock cover is left out of account the problem reduces itself to that of designing the pipe to give strength characteristics similar to those necessary in the open.

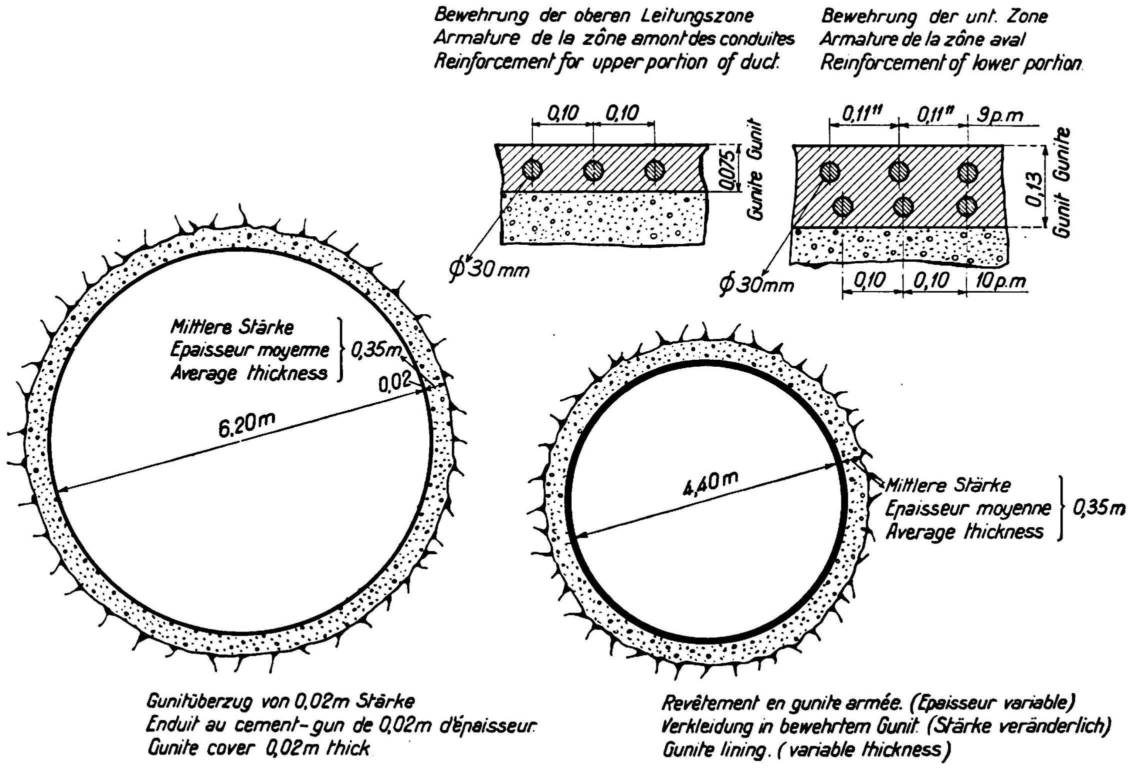


Fig. 3 and 4.

Cross section of water duct.

Cross section of pressure duct and details of reinforcement.

To our knowledge, however, no examples exist of reinforced concrete pipes even remotely approaching the diameter and pressure that occur here. The present state of knowledge on reinforced concrete pipes is such that engineers deem it advisable to observe the following two fundamental rules:

1) The reinforcement must be given a cross section large enough to withstand the whole of the pressure, and
2) The concrete must be given a thickness such that, taking into account the presence of the reinforcement, its limit of resistance to extension will not be exceeded.

The latter rule is of essential importance because a pipe becomes practically useless if cracked.

To apply both these rules in the present case would have meant providing 200 cm^2 of steel per metre run of the pipe and making the pipe wall at least

1 m thick. But in a lining of that thickness would the fundamental rules governing the resistance of reinforced concrete to direct tension still hold good? It seems unlikely. Despite the magnitude and cost of the work carried out in that form there could be no certainty that its permanence would not be imperilled by serious cracking or might not necessitate onerous measures of consolidation soon after the plant had been brought into operation.

If the idea of a reinforced concrete pipe were to be abandoned the only solution on accepted lines would be to extend the steel pipe for some 30 m length inside the tunnel, and this might be done in two ways. The first alternative would have been to arrange the pipe freely within a tunnel large enough to allow access to its outside surface — but the amount of excavation involved would have been enormous and it would have been necessary to provide a lining against falls of rock. The second alternative would have been the more usual one of filling the space between the pipe and the rock with concrete, an awkward and costly proceeding. Whichever method had been adopted the cost of the work would have attained large figures.

Economic considerations led to another solution namely the construction of a pipe formed of ordinary concrete hooped with steel cables.

II. Preliminary investigations. Experiments and tests.

One of the greatest of the difficulties needing solution was that of how to form the hooping underground. As soon as the scheme was considered it was realised that the hooping would necessarily have to be made as follows (Fig. 5): when the full section of the tunnel had been excavated the cables would be placed in circular tubes against the rock wall; then the pipe would be formed by concreting up to the rock, burying the tubes in the concrete; after a few days for the latter to harden, the cable would be tensioned and secured, the tube would be filled with cement grout to preserve the metal, and an injection would be made between the concrete and the rock so as to increase the factor of safety by making the rock participate in the strength of the structure.

The main difficulty was to decide how to attach the cables for the purpose of putting them in tension. The first scheme (Fig. 6) was to have a side tunnel giving access to one end of the cable while the other was embedded in the concrete: but with this arrangement the tension in the cable would fall off progressively from the end actuated by the jacks to the other end, by reason of the friction between the cable and the sheath.

Experiments were then made to determine the coefficient of friction between cable and steel plate, and to try to find substances which would reduce the coefficient as much as possible. No such material was found which would reduce the coefficient of friction below 0.10—0.15, and assuming the value was 0.15 the tension at the sealed end of the cable would amount only to the fraction $\frac{1}{e^{2\pi \cdot 0.15}} = 0.385$ or about one-third of the tension imposed by the jacks at the other end. Such unevenness of tension would not have the effect of producing a flexure liable to crack the pipe: the pressure curve would still remain very well centred and would deviate from the mean fibre only by very small amounts: nevertheless it is true to say that very poor use would thus be made of the cable. To improve the arrangement it would be necessary at least to leave both ends of the cable free so that both could be jacked and the length of the surface under friction reduced to a semi-circumference, and if this were

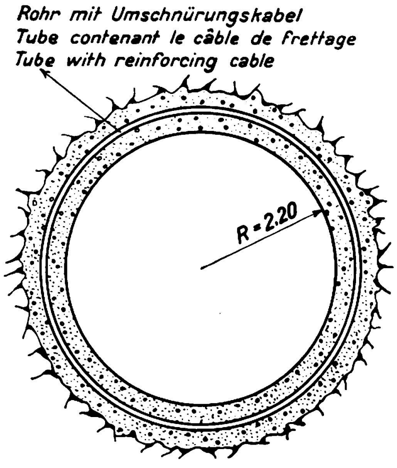


Fig. 5.

Diagrammatical section of duct with lateral reinforcement.

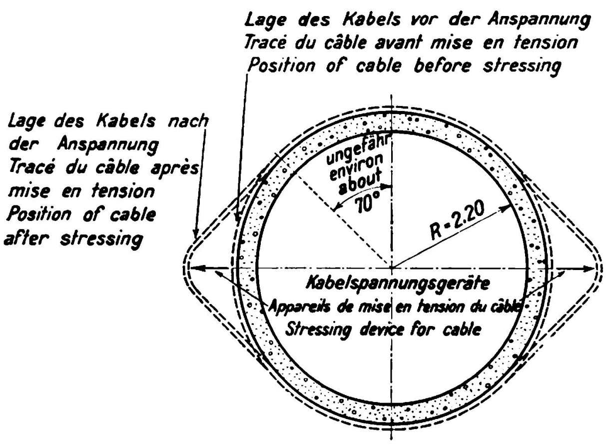


Fig. 7.

Diagram of second proposal for lateral reinforcement.

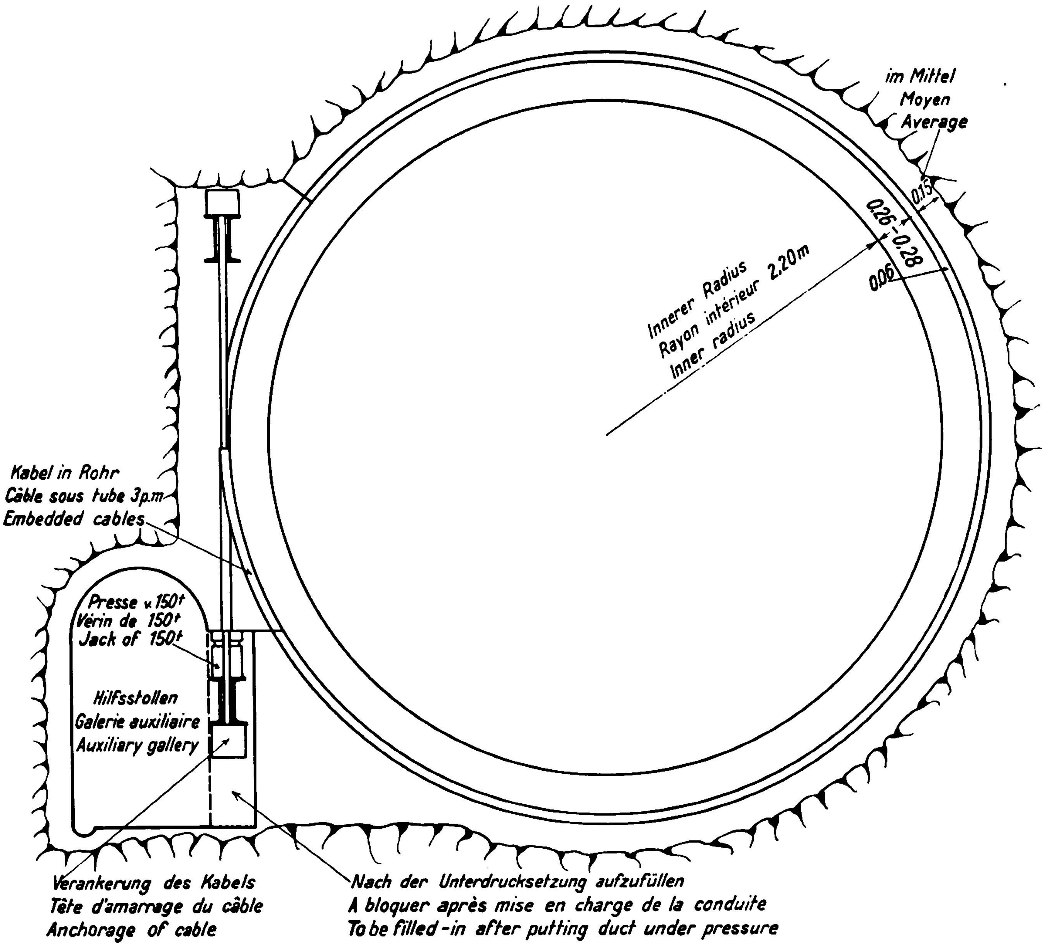


Fig. 6.

Proposed method of reinforcing underground duct by stressed steel cables.

done the ratio between the greatest and least tension in the cable would amount to $\frac{1}{e^{\pi \cdot 0.15}} = 0.62$, corresponding to a loss of 38%.

It should be possible to go further in this direction by making use of graphites to lubricate the cable, but this research was not followed up as it was supplanted an especially interesting suggestion made to us at that juncture by Mr. Guerrier, engineer to the firm of Léon Ballot. This proposal has enabled us to carry out the hooping operation in a very practical way whereby the coefficient of friction becomes a matter of entirely secondary importance.

Mr. Guerrier's idea was to tension the cable by deforming its shape as shown in Fig. 7. This device enables the jacks to be placed inside the pipe itself, so avoiding the need for the auxiliary tunnel. The effect of the friction of the cable on the sheath is reduced to a part of the circumference subtending an angle of about 70⁰ at the centre. The simplest arrangement, *a priori*, would appear to be that of making the two jacks press one against the other (Fig. 8),

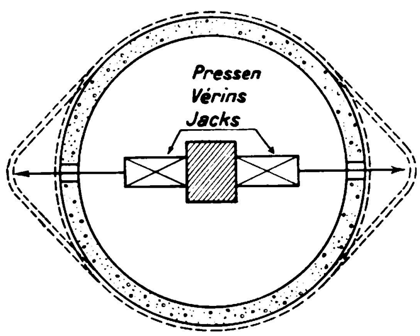


Fig. 8.
Scheme.

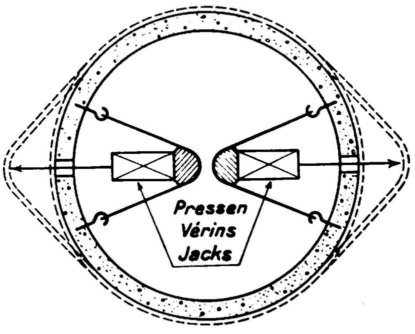


Fig. 9.
Adopted scheme.

but as the cable would be bearing on only a portion of the circumference there would be a tendency to ovalisation and the line of pressure would deviate considerably from the mean fibre. To re-establish a state of elastic equilibrium with the pressure curve close to the mean fibre it might be suggested that tie bars should be added, and the stress in them carefully adjusted at each moment of the operation; this, however, would be a very delicate matter, and it did not appear a practicable solution. It was deemed preferable to adopt the arrangement in which the two jacks are left independent of one another, each bearing upon the pipe itself through the medium of cables sealed in as shown in Fig. 9.

Once this essential point had been decided the hooping scheme took definite shape as indicated in Fig. 10. The protecting tube for the cable is enlayed at each end of the horizontal diameter, in the shape of a flat box which enables the cable to be deformed as requisite for tensioning. At the bottom the two ends of the cable are provided with cast steel anchoring shoes, embedded in the concrete and pressing against one another through the intervening concrete. Where the cable passes through the side boxes it is exposed through openings provided for the purpose, and the jacks operate on the cable through the medium of pieces of cast steel called mushrooms, which are enclosed in the boxes and serve to give the deformed part of the cable a large enough radius to avoid breaking the strands by bending. The jacks bear against the pipe through anchoring irons enclosing the boxes. All these arrangements will be described in greater detail later.

The proposal as thus conceived had some rather bold features which made it inexpedient to put the scheme into full effect until qualms had been allayed by preliminary experiments.

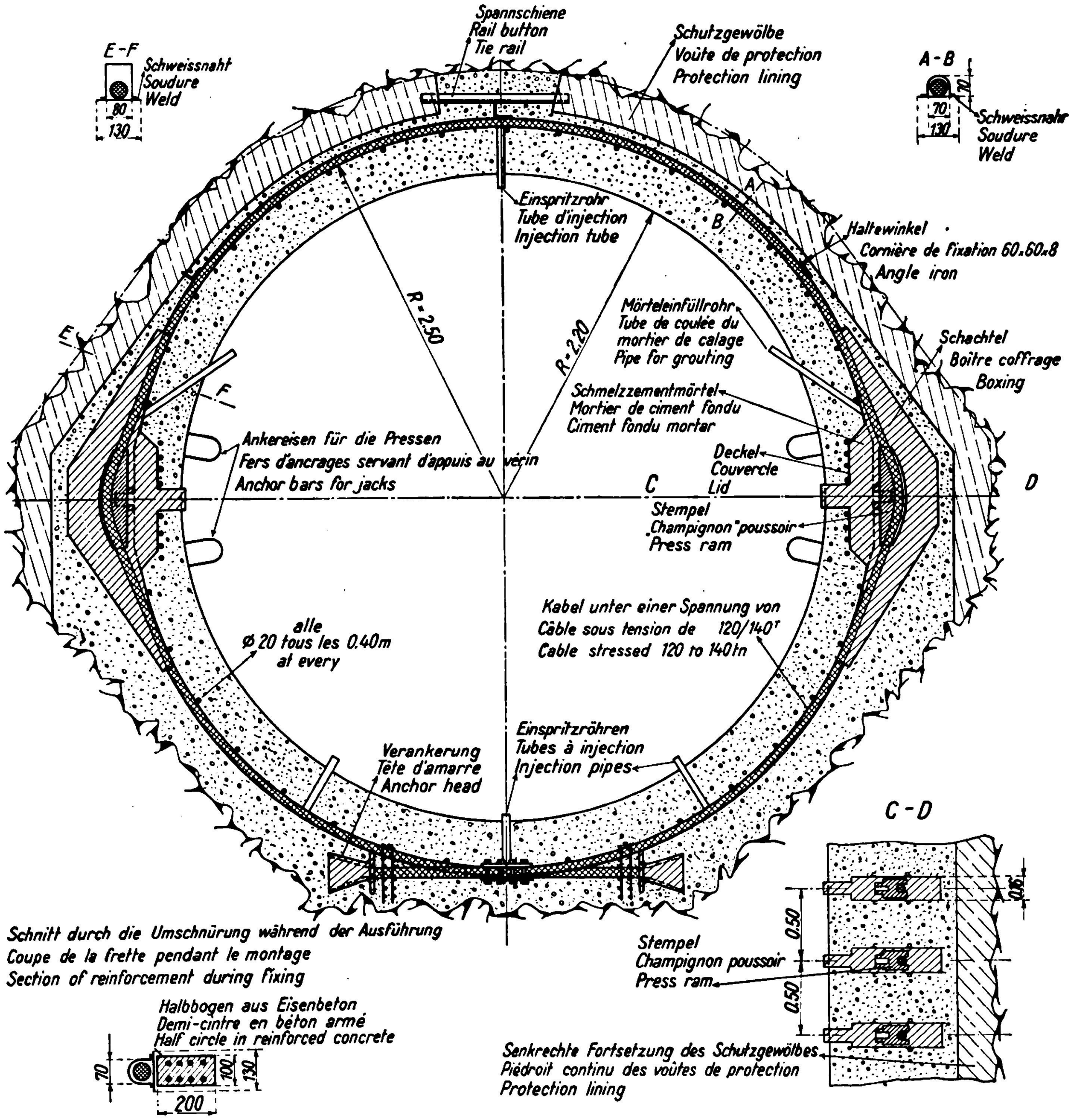


Fig. 10.

Pressure ducts. Laterally reinforced partion. Section showing lateral reinforcement.

In the first place, the amount and bulk of material that had to be gathered around each cable made it necessary to reduce the number of cables per metre run of piping to a minimum. Various considerations contributed to the decision to fix this number at two, each cable then having to receive a tension of at least 110 metric tons. What would be the behaviour of a concrete pipe subjected to localised loads of this magnitude? Nothing but experience could answer this.

Secondly, theory indicated that the curve of pressure would remain very close to the mean fibre in spite of the localisation of forces resulting from the cable being actuated by the jacks at only two points in the circumference: but the fact called for experimental verification.

Finally, experiment alone could determine what would happen to the concrete under the enormous local pressure brought to bear by the cable, and to what extent the cable would be impressed into the concrete.

It was decided, therefore, to carry out a full scale experiment on a vertical cylinder of 4.40 m diameter, 25 cm thick, very lightly reinforced. The essential conclusions reached in this way were the following:

1) Only one crack was observed; this happened when the tension in the cable amounted to 125 metric tons and it took the form of a complete circle on the

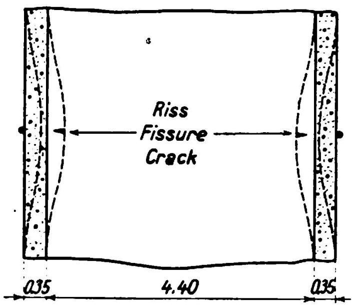


Fig. 11.

Longitudinal deformation of tube with local lateral binding.

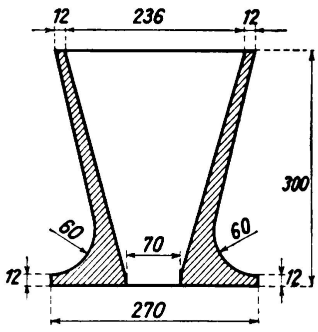


Fig. 12.

Section through anchor shoe.

inside surface of the pipe within the cable. It was caused by longitudinal bending of the walls (Fig. 11), and the inference was drawn that if, under the conditions of the experiment, the tension of the cables is to be carried beyond 125 tonnes, care must be taken not to impose this load in a single cable alone but to do so in several adjacent cables at the same time. It will at once be clear that if a cable has been stressed up to, say, 120 metric tons without cracking, the tensile forces which tend to produce cracking will be greatly lessened as the neighbouring cables are tensioned, so that when the hooping of the whole length of pipe has been completed there will no longer remain any danger of cracking subsequently taking place. It may be added that actually such cracking due to bending is of no importance as regards the quality of the pipe; nevertheless its occurrence has been avoided.

2) The tension in the cable was increased up to 156 metric tons without any other crack appearing. It is noteworthy that there was no longitudinal crack, such as could not fail to have arisen if the curve of pressure had come outside the middle third of the section.

3) The measuring instruments fitted on the surface (the acoustic controls of Mr. Coyne) showed that in a longitudinal direction the stress in the concrete was distributed over a length of about 1 m to each side of the cable, even though the thickness of the wall was only 0.25 m. This result served to allay any possible apprehension regarding the actual scheme, in which the cables were

to be spaced 0.50 m apart in a minimum thickness of wall of 0.40 m, by making it clear that in spite of the hooping being concentrated in a small number of cables under heavy tension the compression in the concrete would be practically uniform.

4) Finally, it could be ascertained that the cable and its sheath left no perceptible impression on the surface of the concrete.

The experiments having been entirely successful it was decided to proceed with the application of the scheme over a length of 28.5 m at the bottom end of each of the pressure conduits, making 114 m in all.

Other experiments were also carried out in order to determine certain details of the design. Among the first of the problems arising was that of finding an economical means of fixing the ends of the cable. One *a priori* possibility was simply to embed the two ends in the concrete of the pipe itself, relying on adhesion to give the necessary anchorage. It would be necessary for this purpose to completely unstand a certain length of each end of the cable, for it is not possible to rely on the adhesion of a cable or a stand, the effect of tension being to reduce the diameter by a perceptible amount. So many interlacing wires would, however, have made concreting difficult, and it was deemed preferable to terminate the cable by means of ferrules such as are used in the construction of suspension bridges. It was found by trial that cast steel ferrules of 30 kg weight (Fig. 12) were amply sufficient to carry a tensile force of over 220 metric tons no case of breakage of a ferrule occurred, nor was there any case of breakdown in the adhesion of the wires. The cable always broke on reaching its normal breaking load; moreover it scarcely ever broke at the opening of the ferrule although one might expect it to be slightly weakened there by the curvature of the wires. It may be added that instead of the filling being made with molten metal as is usual in suspension bridges, this was done with cement mortar and the result is entirely satisfactory. Ferrules were also successfully made entirely of reinforced concrete, but the steel type was finally preferred.

Another important problem was that of securing the cables, when tensioned, by the interposition of packing between the steel "mushroom" and the cover of the flat box so as to allow the jacks to be released. It was impossible to insert a previously fashioned body into this space, and there was no alternative but to pour into it some material that would set and would be able after a short time to withstand a compression of over 100 kg/cm². After some unsuccessful experiments in packing with dry sand the choice finally fell on the use of mortar made with ciment fondu, which gave every satisfaction: the jacks could be removed, without danger, six or seven hours after casting, and the cement having been cast in a completely closed box, bound by some means in all directions, underwent only an infinitesimal amount of settlement at the moment the jacks were removed — less than a millimetre, whereas the extension of the cable itself through tensioning is about 13 cm.

Further tests were made on the cables to determine their elastic elongation and permanent set, particularly for the purpose of calculating what depth should be given to the flat boxes. These tests showed that it was possible to rely on the strain being nearly elastic, with a coefficient of elasticity equal to about half that of the steel.

Finally, mention should be made of the measurements of loss of tension in the cable due to ageing; these showed that such losses could not be considerable, and the only action taken in consequence was to increase the initial tension to 135 tons instead of the 110 tons strictly necessary.

III. Detailed description of the arrangement.

In this chapter it is proposed to review all the details of the scheme, giving briefly the reasons which led to their choice:

1) *Cables.* — As stated in the last paragraph of the preceding chapter, the cables were tensioned to 135 tonnes. This led to the adoption of cables having a breaking strength of 220 metric tons. The working stress, being close to the elastic limit, may appear high by comparison with the stresses that are usual in constructional work. Actually when the cable is tensioned the stress is adjusted to a figure which is accurately known; the experiment on the hooping of a vertical cylinder had shown that the cable did not, at any point, undergo such flexure as might reduce its breaking strength, so that no fear need be entertained

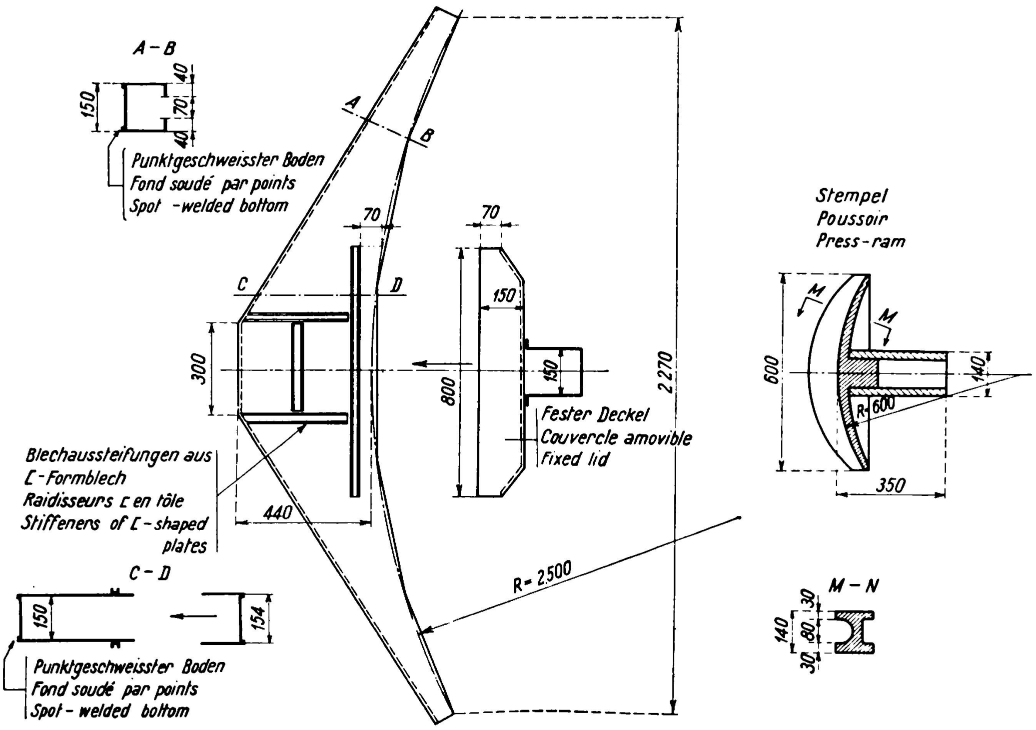


Fig. 13.

Pressure ducts. Steel box for placing and fixing of lateral reinforcement.

of the real stress exceeding that calculated and there is no point in preserving an exaggerated margin of safety in the operation. Moreover, in the moment immediately following the tensioning and packing of the cable there is bound to be a tendency for the tension to diminish. Once the packing has been done, the compressed ring of concrete and the cable act together as one; under the action of the internal water pressure they undergo equal amounts of elongation — but these amounts are very small; the static pressure produces an additional tension in the cable which may be put at 3 tons, and the hammer blow a further one ton which develops progressively during the period of closure of the valves, about 4 seconds. These extra tensions, which are added to the initial tension of 135 tons, are the only forces in the problem imperfectly known, and in respect of these there is a very high factor of safety. The cables consist of six strands of 19 bare wires of 4.15 mm diameter and 130 kg/mm² breaking strength.

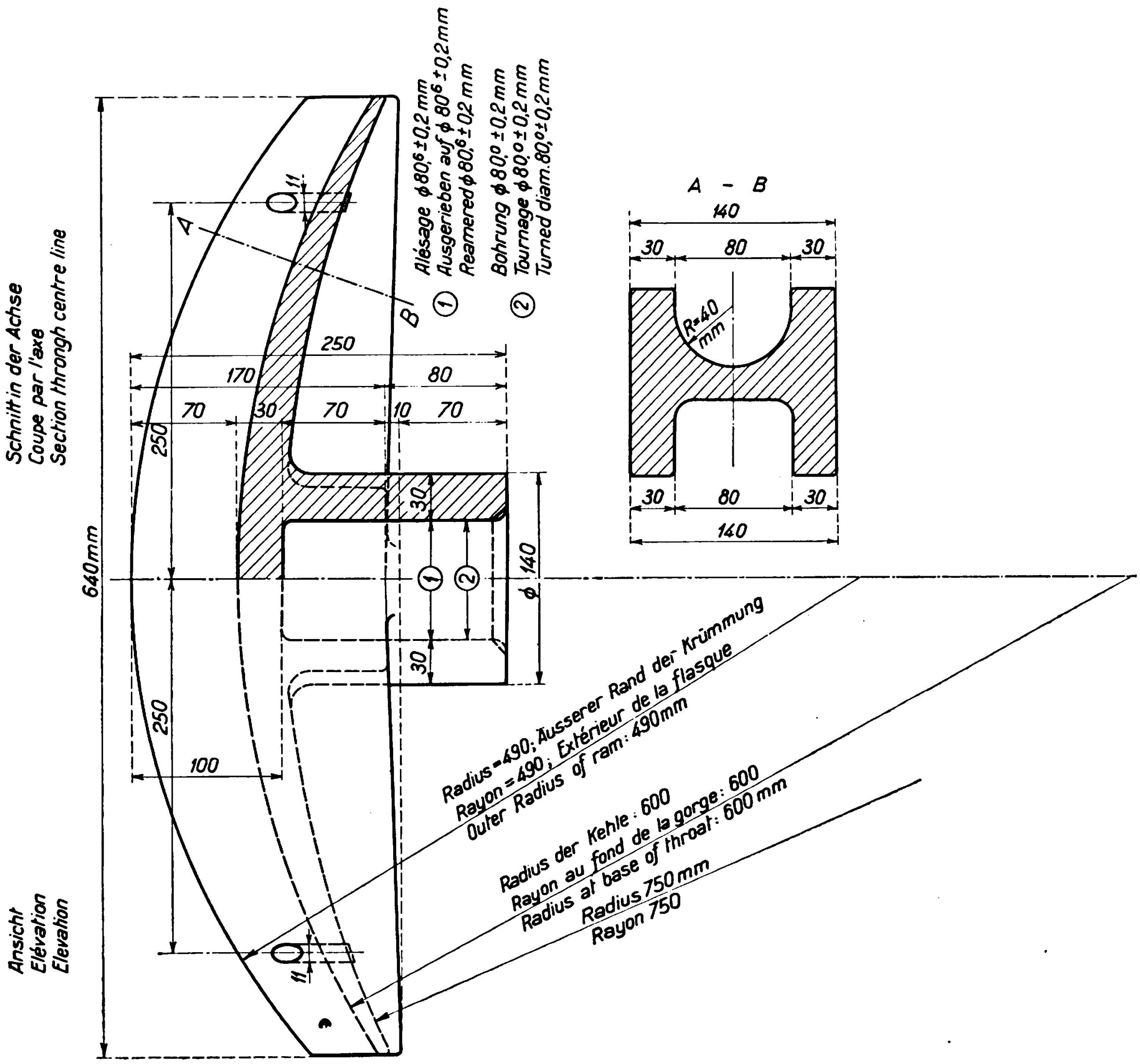


Fig. 14.

Lateral reinforcement of pressure ducts. Presse ram.

2) *Anchoring ferrules.* Fig. 12, already commented upon in chapter II, gives a clear idea of the solution adopted.

3) *Tubes and flat boxes.* The protecting member for the cable includes three lengths of tubing and two flat boxes. For convenience in fabrication (of which details are given later) each length of tube has been formed in two parts: a half tube of Ω - section and a closing plate. The half tube is bent to a radius of 2.50 m. The flat boxes, the object of which is to allow of deforming the cable, are built up as shown in Fig. 13 and consist of the box itself, and the lid,

the latter being furnished with a short tube which forms an orifice for access to the cable after concreting.

4) *Mushrooms.* The cast steel mushroom (Fig. 16) is housed inside the box, and it is through this that the jack acts upon the cable; its object is to impose a suitable radius of curvature on the latter. The mortar poured between the mushroom and the lid of the box forms the packing which allows the jacks to be removed.

5) *Reinforcements.* No circular reinforcement is provided. A few bars of 20 mm diameter spaced at 0.40 m form a very light longitudinal reinforcement, which is in fact considered to be superfluous in view of the fact that the extent of the longitudinal stress for each hoop is known. At the downstream end of

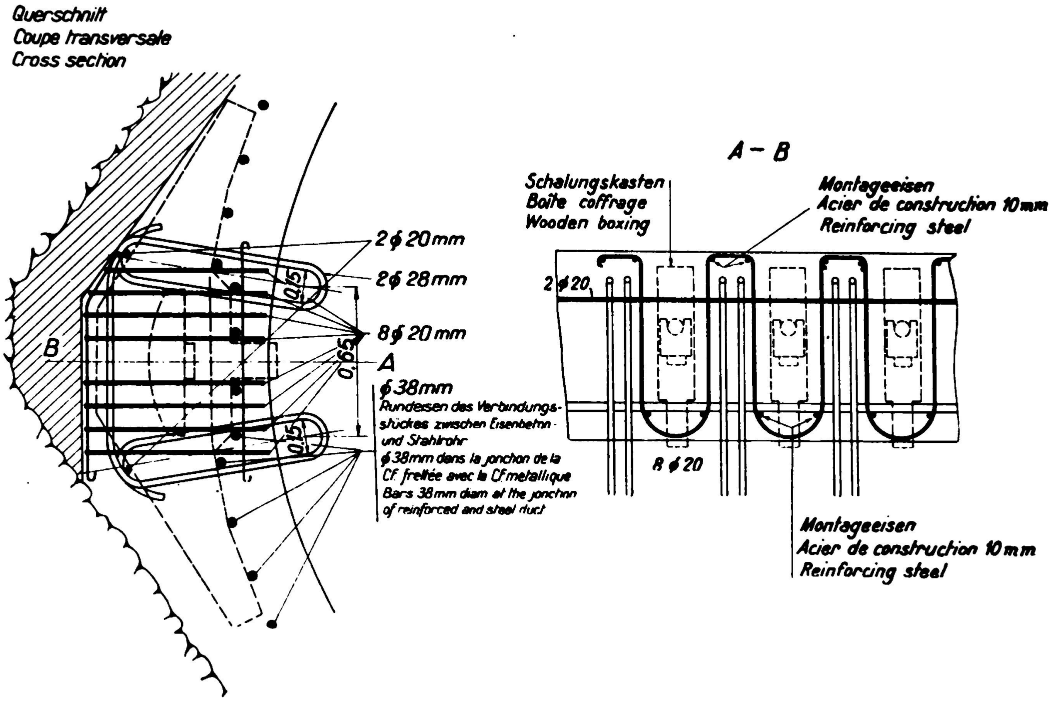


Fig. 15.

Pressure ducts. Laterally reinforced portion.
Reinforcement round for wooden boxing read steel boxing press rams.

the hooped construction the longitudinal reinfarcement is considerably strengthened through the presence of the anchorage bars of the steel piping.

The most important of the steel parts are those required on the one hand to anchor the jacks and on the other hand to take up the permanent reaction of the cable, which, to the right of the flat boxes, is exerted on the small thickness of concrete between the lids of the boxes and the intrados of the pressure pipe. Fig. 15 shows the arrangement adopted, including also the longitudinal bars of 38 mm diameter serving as anchorage for the steel piping.

At the centre of each space between consecutive cables are placed two stirrups of 28 mm diameter, made from a single length of bar in two parallel arms, 7 cm apart. These form two loops of 150 mm diameter projecting inside the tunnel, at levels respectively 0.325 m above and below the horizontal axis of the conduit. In this way the jack for tensioning the cable can bear on four double loops, each of which, under normal stress, is capable of withstanding a tensile load of 30 tons.

The pressure of 100 to 120 tons exerted by the mushroom, after packing, against the strip of concrete enclosing the lid of the box, is distributed

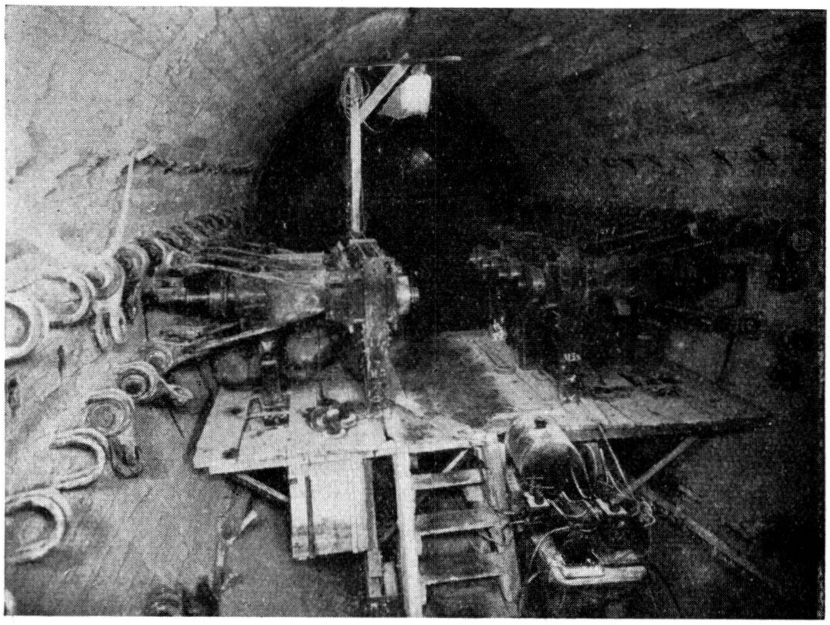


Fig. 16.
Movable stand for presses.

throughout the thickness of the pressure pipe by horizontal bars uniformly curved to a large radius so as to bind the said strip.

Finally, the front of each anchoring ferrule on the end of a cable is pro-

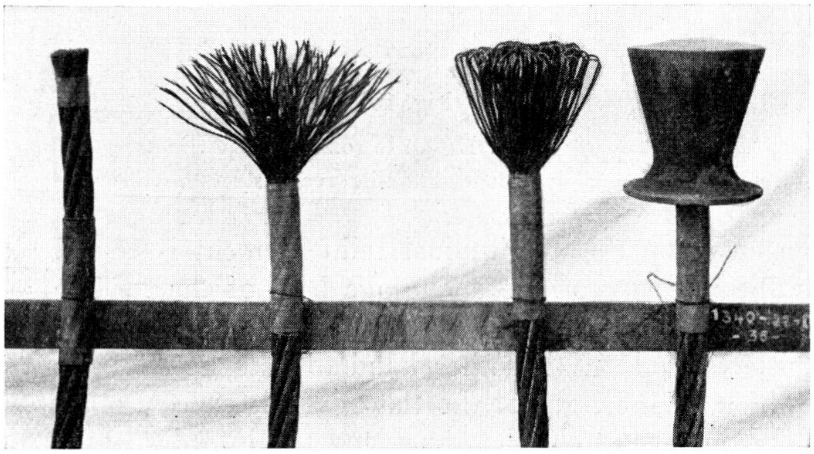


Fig. 17.
Preparation of rope anchors.

vided with a double grillage of 8 mm bars with 6 cm mesh, binding the concrete which is heavily compressed by the ferrule (Fig. 10).

6) *Trolley for jacks.* The jacks are carried on trolleys which run on a temporary track. Two such trolleys were used, one carrying two jacks and the other

four (Fig. 16). Each jack stands on a metal base carried by the trolley but is free to move 10 to 15 mm in all directions relatively to the base. Passing through the base are two spindles of 110 mm diameter, screwed and provided with nuts; these are situated 25 cm on each side of the jack, in the same horizontal plane. On each spindle is a loose pulley over which is passed a cable with a hook at either end, enabling it to be attached with the aid of bars, etc. to the anchoring hooks described in paragraph 5 above.

When not in action the jacks are carried by a screw centering device which enables them to be rapidly aligned with the axes of the orifices giving access

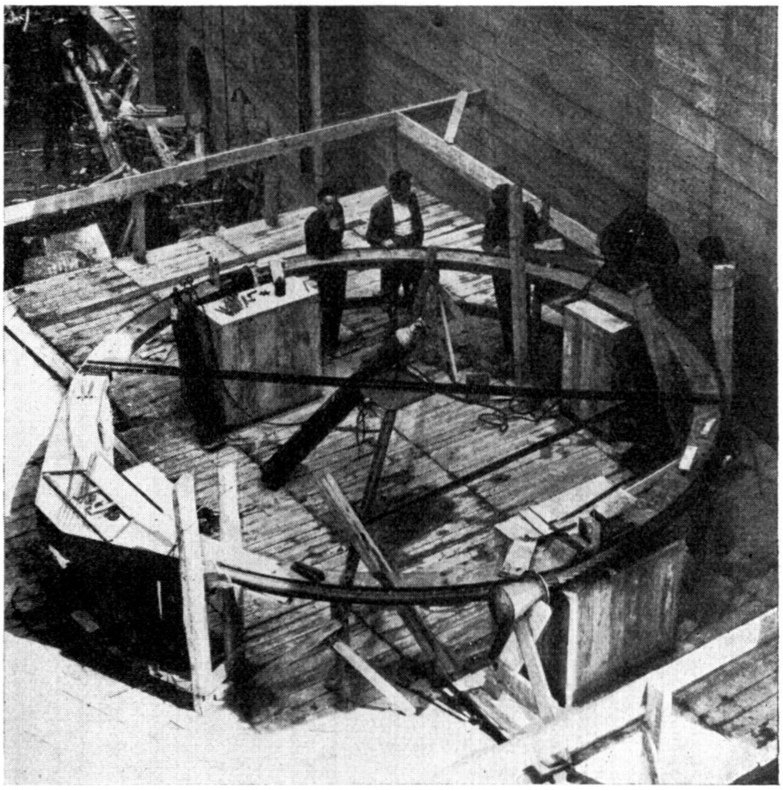


Fig. 18.

Preparation of reinforcing hoop.

to the cables. When in action they are borne on the anchorages, and the play provided in the base ensures that the trolley shall be unaffected by the forces developed.

IV. Construction of the hooping.

1) Preparation of the hooping cables. The cables are supplied in lengths of 18.5 m, this being sufficient to form one hoop. They are bound at a distance of 32 cm from each end. An anchoring ferrule is slipped over each end of the cable, and the latter is unstranded into wires hooked at their ends. The tuft so obtained is sealed into the ferrule with mortar made by mixing 50 kg of ciment fondu with 50 kg of sand of 0—5 mm gauge. Fig. 17 shows the stages of this work.

2) *Preparation of the hoops.* Mr. Pfaff, Engineer of Public Works, to whom all the arrangements for the hooping are due, conceived the idea of forming the hoops into rigid shapes before transporting them underground rather than assembling the cables, sheaths and boxes on the spot, which would have

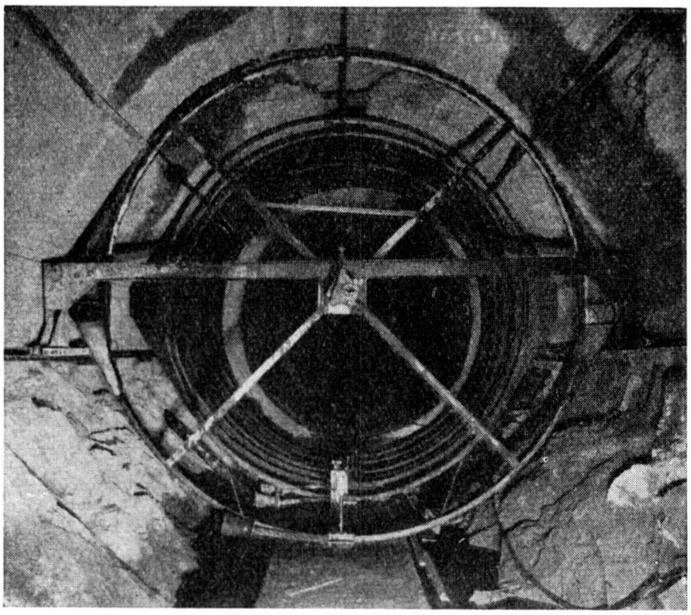


Fig. 19.
Placing reinforcing hoops into position.

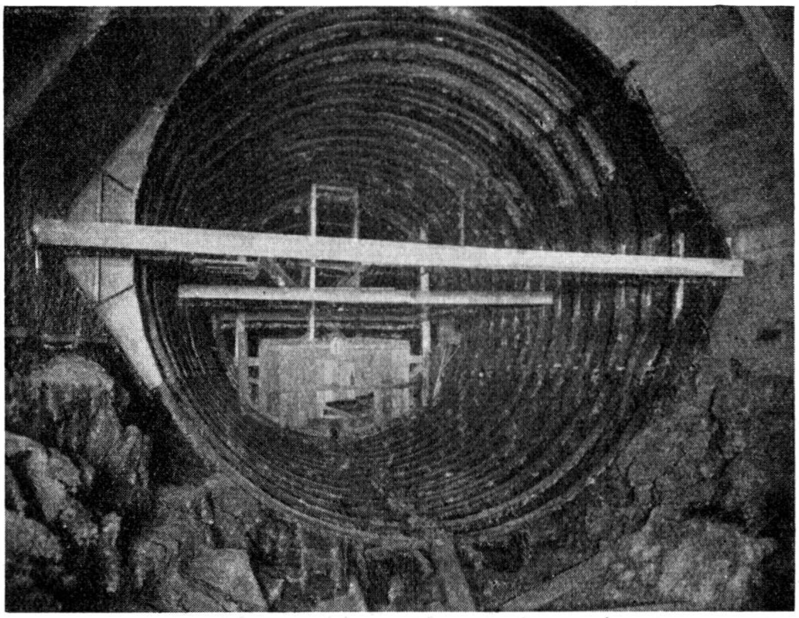


Fig. 20.
Reinforcing hoops after placing into position.

been very awkward to do. For this purpose he built, outside the tunnel, an assembly platform of two moveable half-centreings of reinforced concrete made with ciment fondu, to a radius of 2.5 m, arranged horizontally 1 m above ground level (Fig. 18).

The cable is first fixed around this centreing and is temporarily placed on the packings; then the three portions of closing plate for the sheath are inserted between the cable and the centreing; next the two ends of the cable are pressed together at their junction with the aid of a clamp; two screw jacks

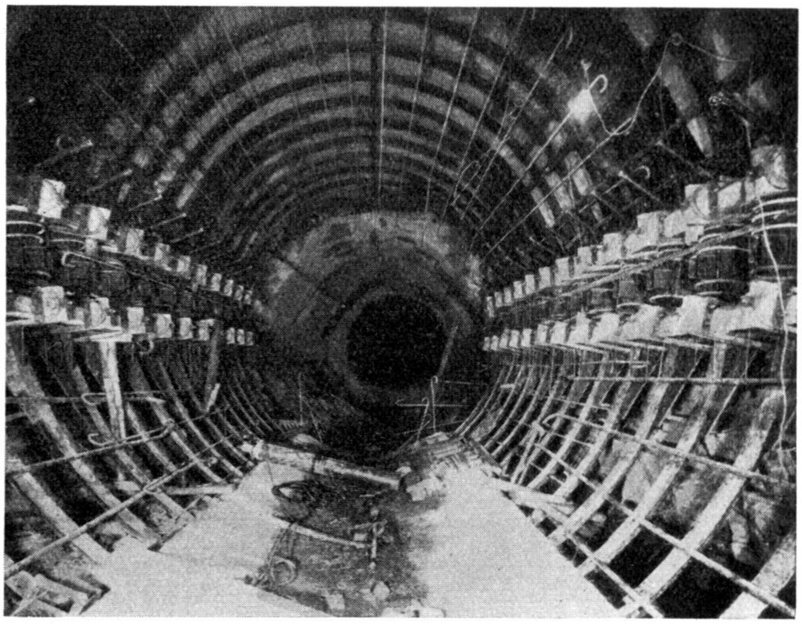


Fig. 21.

Duct before concreting.

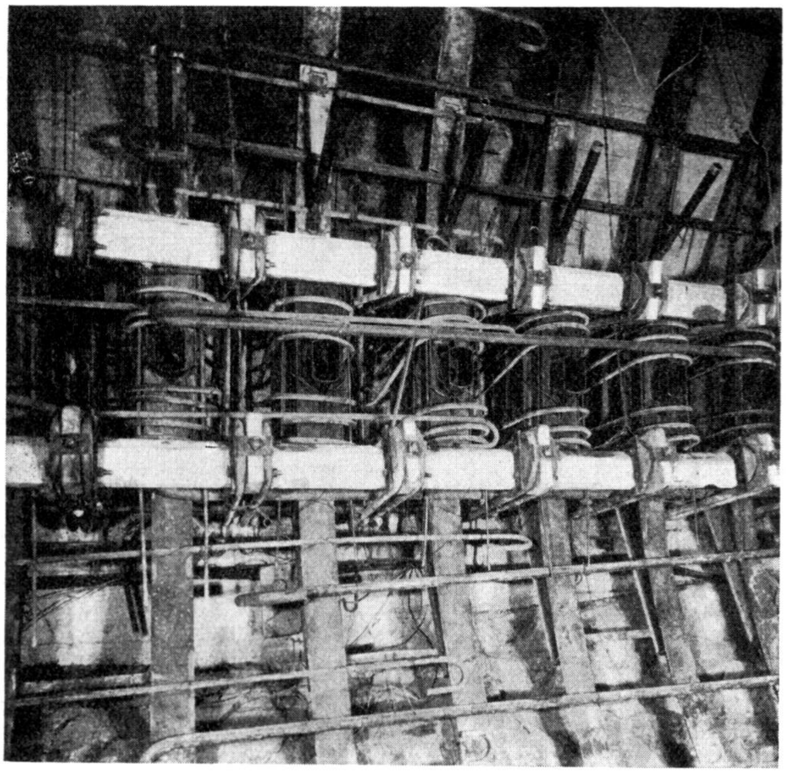


Fig. 22.

Centreing boxes and reinforcement before concreting.

between the ends of the half-centreings lightly stretch the cable and give it a diameter of exactly 5.0 m. Finally the Ω-shaped sheaths and the boxes without their lids are placed in position and are spot welded to the closing plate.

At least three hours were required to prepare a hoop, and the error in its diameter is only a few millimetres.

3) *Erection of the hoops.* The rigid whole formed in this way, some 600 kg in weight, has to be transported and erected in the tunnel, the cross section of which is barely larger than that of the hoops. It is lifted by a crane and transported on a trolley which runs on two side tracks (Fig. 19); it is then adjusted into place and attached to rows of angles sealed into the rock so as to avoid later displacements (Fig. 20).

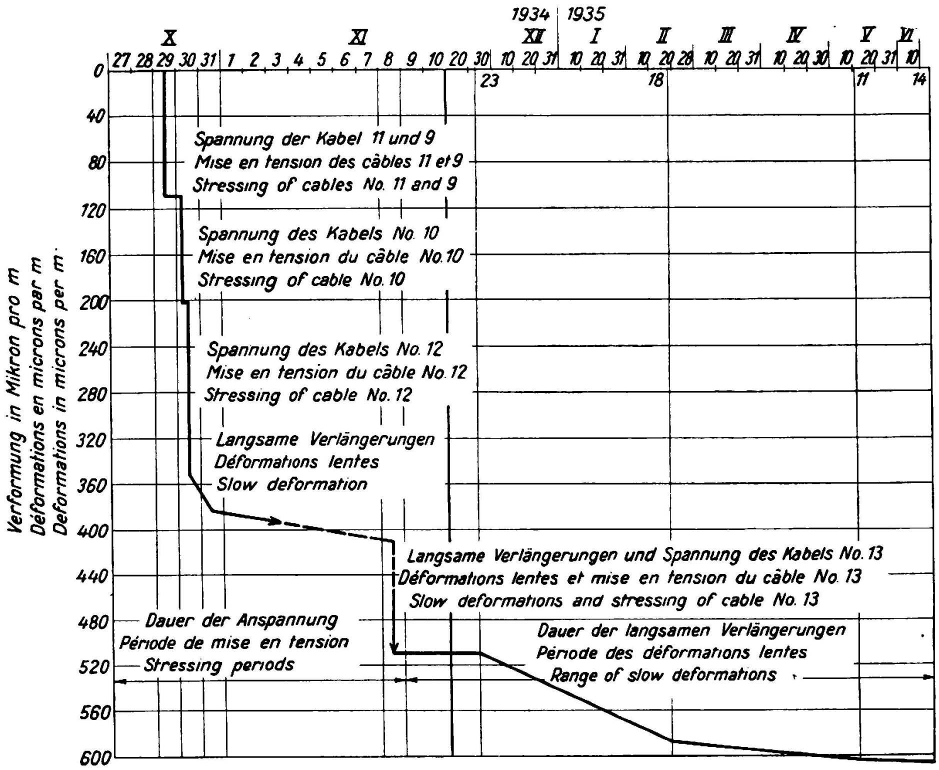


Fig. 23a.

First period.

Stressing of cables 9 to 13 (from 29th October till 8th November 1934) and slow deformations (until 14th June 1935).

4) *Sequence of operations in the construction.* The lower portion of the pipe is concreted first and the mushrooms, covers of boxes and reinforcements are next added in that order. Figs. 21 and 22 show the appearance of the pipe after these operations. The concreting is then carried out. The concrete was made from crushed gravel of 70 mm gauge, except in the reinforced parts where it was necessary to reduce the gauge to 30 mm. The mixture adopted contained 400 kg of "iron Portland" cement per m^3.

5) *Tensioning and packing the hoops.* The tensioning of the hoops is undertaken at the end of a fortnight allowed for hardening (Fig. 16). The trolley

carrying the jacks is brought opposite the cable to be tensioned and each jack is aligned with the orifice in the lid of the flat box; a thrust bar is inserted between the mushroom and the ram of the jack, and the jack is packed in position. The side slings are hooked to the anchoring loops and are lightly stretched by means of the nuts on the threaded spindles.

Oil under the appropriate pressure for tensioning the cable is then forced into the jacks by starting up a small electric pump which feeds two opposed pistons simultaneously. At any time the amount of tension in the cable can be found from a table, being expressed as a function of the pressure exerted by the jack, and of the deformation of the cable as measured by the movement of the mushroom transmitting the pressure. When the tension in the cable

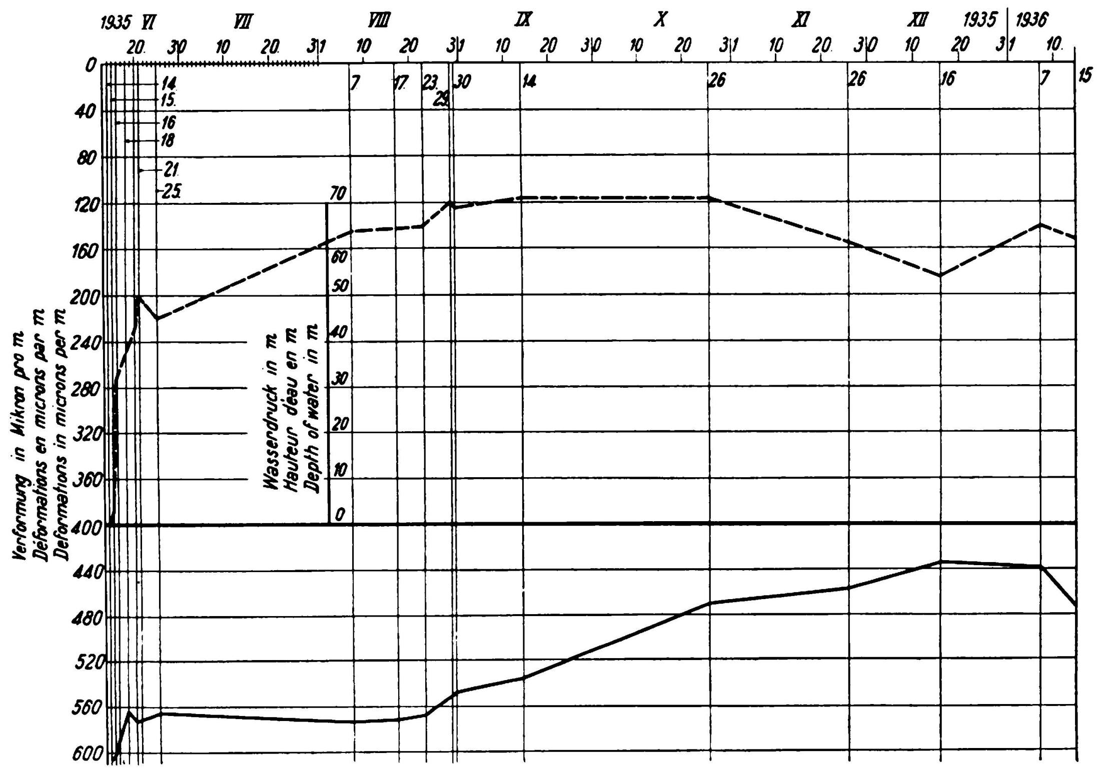


Fig. 23b.

Second period. Duct under water pressure. Deformation diagram of duct N° 1 (measured by acoustic methods).

reaches a value of about 130 tons the jacks are locked by their safety nuts and the flat boxes are filled with mortar containing $^1/_3$ ciment fondu. After about seven hours the mortar is hard enough to withstand the reaction of the cable, which is of the order of 100 tons. The jacks are then removed with due precaution and passed to the next cable. It may be added that the movement of the mushroom is about 13 cm and the drop back on taking out the packing is only about 1 mm which releases only a negligible proportion of the tension in the cable.

The whole tensioning operation does not take longer than 12 hours, so that working continuously it is possible to stretch two cables a day with one pair of jacks.

6) *Finishing operations on the pipe.* The tubes are then filled with cement by injection in order to preserve the cable, and injections are also made into the junction between concrete and rock and into the rock cleavages. Finally the anchoring hooks are removed and the inside of the pipe is covered with the cement gun.

V. Control of the hooping work. Control in Service.

A considerable number of the accoustic controls invented by Mr. Coyne have been applied to the thickness of the concrete lining for the purpose of measuring the state of compression therein and of ensuring the maintenance of the proper condition in service.

Fig. 23 gives, as an example, a picture of the state of compression in the concrete between cables Nos. 11 and 12 of pipe No. 1. In this graph three periods may be distinguished:

1) The period during which the cables adjoining the zone in which the acoustic device is fitted were being tensioned (from October 27th to November 8th, 1934): here the curve clearly shows the changes in the compressive stress of the concrete at the times of tensioning the five cables numbered 9—10—11—12—13 which bracket the point in question.

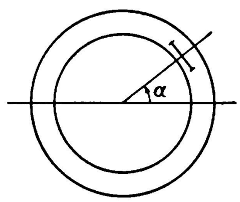
These readings fully confirm what had been established by the preliminary experiment, namely that the zone of influence of any one cable extends over more than one metre on each side, giving the assurance that the pipe is compressed very uniformly despite the considerable spacing of the hoops. The deformation at the end of this period amounts to 500 microns per metre, and if, as the concrete is not very old, the modulus of elasticity be assumed at 150,000 kg/cm^2 this corresponds to a stress of 75 kg/cm^2.

2) The period of slow deformations (from November 8th, 1934 to June 14th, 1935): herein the concrete is continuing to deform under the sustained action of the loads applied to it. The plastic strain exceeds 100 microns per m.

3) The period in which the pipe first became filled with water (from June 14th, 1935): under the action of the internal water pressure part of the compression in the concrete is released. Under a hydraulic head of 71 m the decompression amounts to 135 microns per m. If the pipe were free, without the rock adding to the resistance, the decompression would be about 300 microns per m.

In Fig. 23 it is possible to perceive a highly satisfactory parellelism between variations in the compression of the concrete and variations in the water pressure. Such small inconsistencies as are present in the curve may almost certainly be attributed mainly to variations in the temperature of the water, which causes a slight error in reading the acoustic controls.

If, now, a comparison is made of the compressive stresses and amounts of decompression obtained at the different points tested — taking account of the variations in thickness of the pipe — a satisfactory agreement between the results is seen to be present as indicated in the following table:

No. of control	Nos. of cables on each side of the control	Position of the control in the section α (1)	Compressive stress after hooping	Residual compressive stress under a head of 71 m of water	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
1	0—1	$\alpha = 60^\circ$	300	170	(1) meaning of α:
3	7—8	$\alpha = 90^\circ$	580	440	
4	11	$\alpha = 135^\circ$	465	360	
10	11—12	$\alpha = 120^\circ$	530	420	
11	11—12	$\alpha = 60^\circ$	605	470	
13	12—13	$\alpha = 0^\circ$	450	220	
14	14—15	$\alpha = 90^\circ$	270	170	
18	38—39	$\alpha = 90^\circ$	470	345	

Summary.

The system of hooping described in this note is a new application of the general constructional method which consists of subjecting the work to pre-established strains in order to improve the distribution of those stresses which will ultimately come to bear. The clever ways in which this principle has already been applied, notably by Mr. Freyssinet and Mr. Coyne, are well known.

Very special difficulties were encountered in the present instance, owing to the work being carried out underground, and it follows that the method can be much more readily applied, under incomparably simpler conditions, for the hooping of concrete pipes in the open air. We are of opinion that it is a method capable of many applications, especially for pipes of large diameter.

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